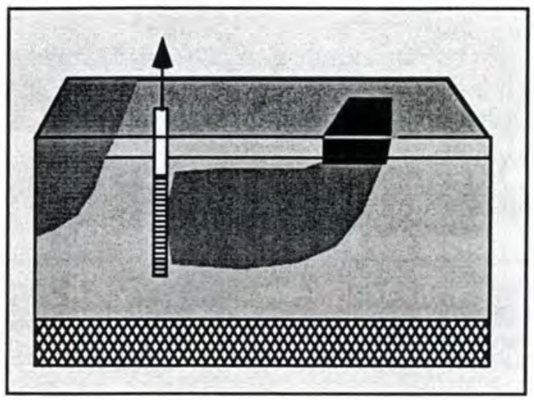
SOLUTIA - 055

HYDRAULIC CONTAINMENT STUDY

Site R
W.G. Krummrich Plant

Sauget, Illinois



PRELIMINARY

Submitted to
Solutia Inc.

June 20, 2001

Groundwater Services, Inc.
2211 Norfolk, Suite 1000, Houston, Texas 77098

GROUNDWATER SERVICES, INC.

EXECUTIVE SUMMARY

Site R is located in an area referred to as the American Bottoms on the east bank of the Mississippi River directly downgradient of the W.G. Krummrich Plant. The plume associated with Site R is approximately 2500 ft wide in a line perpendicular to groundwater flow. Higher concentrations are found in the shallow and middle horizon of the water-bearing unit with lower concentrations found in the deep horizon.

The objective of this study was to determine how much pumping and how many pumping wells are required to hydraulically capture dissolved constituents in groundwater underlying Site R before they discharge to the Mississippi River. A numerical groundwater flow model, MODFLOW, and a particle tracking model, MODPATH, were used to address these questions (Figure 1).

Results

Using a particle tracking approach and conservative site assumptions, **1325 gpm of pumping** is required to completely capture the Site R plume and prevent discharge to the Mississippi River under a low river level scenario. A ten-well system requires less pumping (1325 gpm) than a two-well system (1750 gpm) (Figure 2).

This analysis is conservative as it assumes a conservative plume area requiring capture (3100 ft perpendicular to the river) and that all affected groundwater, even groundwater with extremely low concentrations, requires capture.

The effects of changing Mississippi River levels were not considered in the analysis. Additional modeling would be required to determine the effects of changing river level on the pumping rates required for complete hydraulic containment.

INTRODUCTION

As requested by Solutia Inc. (Solutia), Groundwater Services, Inc., (GSI), has completed a study of hydraulic containment options for affected groundwater associated with Site R near the W.G. Krummrich Plant in Sauget, Illinois. This report summarizes the approach and results of the study.

PROJECT BACKGROUND

Site R is located in an area referred to as the American Bottoms on the east bank of the Mississippi River. The geology of the area is described as consisting of unconsolidated valley fill deposits (Cahokia Alluvium) overlying glacial outwash material (Henry Formation). In general, the permeability of the unconsolidated material increases with depth, with the outwash material being comprised of medium- to coarse-grained sand and gravel. The hydrogeologic conceptual model divides the unconsolidated water-bearing unit into three horizons: the shallow horizon (generally 15-30 ft deep), the middle horizon (generally 30-70 ft deep), and the deep horizon (generally 70-110 ft deep). These unconsolidated deposits are underlain by limestone and dolomite bedrock.

Representative constituents associated with Site R include volatile organics such as benzene, chlorobenzene, acetone, and 1,2-dichloroethane and semi-volatile organics such as phenol, 2-chloroaniline, and 2-nitrochlorobenzene. The plume associated with Site R is approximately 2500 ft wide in a line perpendicular to groundwater flow. Higher concentrations are found in the shallow and middle horizon, with lower concentrations found in the deep horizon.

The objective of this study was to determine how much pumping and how many pumping wells are required to hydraulically capture dissolved constituents in groundwater underlying Site R before they discharge to the Mississippi River. A numerical groundwater flow model, MODFLOW, and a particle tracking model, MODPATH, were used to address these questions.

MODEL DESCRIPTION

Two models were employed in this study: The MODFLOW groundwater flow model, developed by the U.S. Geological Survey (McDonald and Harbaugh, 1988) and the MODPATH particle tracking model (Waterloo Hydrogeologic, no date). MODFLOW was used to simulate the movement of groundwater for baseline conditions and for various pumping scenarios, while MODPATH was used to evaluate this movement by tracking the trajectory of "particles" that represent packets of groundwater as they move through an aquifer over time.

Key Model Attributes, Assumptions, And Input Parameters

Key model attributes and assumptions are listed below:

- A finite-difference grid, with 120 ft by 120 ft cells around Site R and cell size gradually increasing with distance from Site R, (Figure 1), was used for modeling the site.

- Three layers were used in the model: an unconfined shallow layer, a convertible confined/unconfined middle layer, and a confined deep layer. The top and bottom elevations of the water-bearing units were derived from available well logs and a cross-sectional map (Geraghty and Miller, date unknown, "Generalized Geologic Cross-Section A-A").

- Using the data from the literature, slug test results, and calibration work, the following hydraulic conductivities were used in the model:

    - Shallow Horizon: $1x10^{-2}$ cm/sec
    - Middle Horizon: $1x10^{-1}$ cm/sec
    - Deep Horizon: $1x10^{-1}$ cm/sec

- Bedrock elevations, obtained by kriging data contained in Bergstrom and Walker (1956) were imported into the model.

- Geologic cross-section data developed by Bergstrom and Walker (1956) and USGS topographic maps were used to develop a simplified geometric vertical river boundary, where a rectangle was used to simulate the river from the western bank to the middle of the river, and a triangle was used to simulate the river from the mid-point to the eastern bank. The riverbed elevation for each river cell used in the model was derived using this simple geometric model.

- A low river level case with a river stage of 380.9 ft MSL was used for the modeling study. The low river stage was selected because it resulted in the highest hydraulic gradient in the aquifer, making hydraulic capture more difficult. River stage information was obtained from Schicht (1965) and Figure 10 of Schicht and Buck (1995).

- The riverbed conductance was assumed to be 3182 ft^2/day based on data developed by Schicht (1965).
- Constant head cells were used in the model to represent the western bluff line (east boundary) of the modeled area. A constant elevation of 405 ft MSL was assigned to the constant head cells based on potentiometric surface information from November 1990 that was presented in Figure 14 of Schicht and Buck (1995).
- A surface infiltration rate of 7.8 inches per year was used in the model to represent infiltration from rainfall (Schicht, 1965).
- A regional pumping center of 4167 gpm was established in the model to represent ongoing highway dewatering projects in the East St. Louis area (Ritchey and Schicht, 1982).
- Wells used for evaluating plume capture were assumed to be screened only in the middle unit.

Modeling Approach

The MODFLOW model was run under steady-state conditions. Because the resulting potentiometric surfaces from the three layers were very similar, the potentiometric surface from the middle horizon was compared to the potentiometric surface reported for November 1990 reported by Schicht and Buck (1995). This comparison indicated that the general shape and values of the predicted potentiometric surface were similar to the reported potentiometric surface (including the cone of depression caused by the highway dewatering system). Therefore the MODFLOW groundwater flow model was considered to yield a reasonable simulation of the aquifer system.

MODPATH was then run to create a visual representation of groundwater flowpaths based on the pumping rates specified for the scenario. Particles were placed along the boundary of the groundwater plume in a fairly dense pattern. To be conservative, a groundwater plume width of 3100 ft was used based on the non-detect line shown on a map of semi-volatile organics developed by Roux Associates, Inc. (Roux Associates, Inc., May 3, 2000, "Total SVOC Concentrations Middle Hydrogeologic Units).

Wells were incorporated into the MODFLOW model, and the pumping rates were then adjusted until nearly all of the groundwater containing the dissolved plume was captured by the recovery wells. The total pumping rate was then recorded, and the process was repeated for each groundwater pumping scenario.

Modeling Results

To completely capture the assumed zone of affected groundwater, at least two wells were needed with a total pumping rate of 1750 gpm. A ten well system reduced the required total pumping rate to 1325 gpm (Figure 2).

Significant pumping was required to capture potentially low-concentration edges of the plume, indicating that most of the mass flux into the river could likely be captured with lower flowrates.

KEY POINT: PUMPING RATE REQUIRED FOR CAPTURE

Using a particle tracking approach and conservative site assumptions, **1325 gpm of pumping** is required to completely capture the Site R plume and prevent discharge to the Mississippi River. A ten-well system requires less pumping (1325 gpm) than a two-well system (1750 gpm).

This analysis is conservative as it assumes a large plume area requiring capture (3100 ft perpendicular to the river) and that all affected groundwater, even groundwater with extremely low concentrations, requires capture.

The effects of changing Mississippi River levels were not considered in the analysis. Additional modeling would be required to determine the effects of changing river level on the pumping rates required for complete hydraulic containment.

REFERENCES

Bergstrom, R.E. and T.R. Walker, 1956. *Groundwater Geology of the East St. Louis Area, Illinois,* Illinois State Geological Survey, Urbana, Illinois.

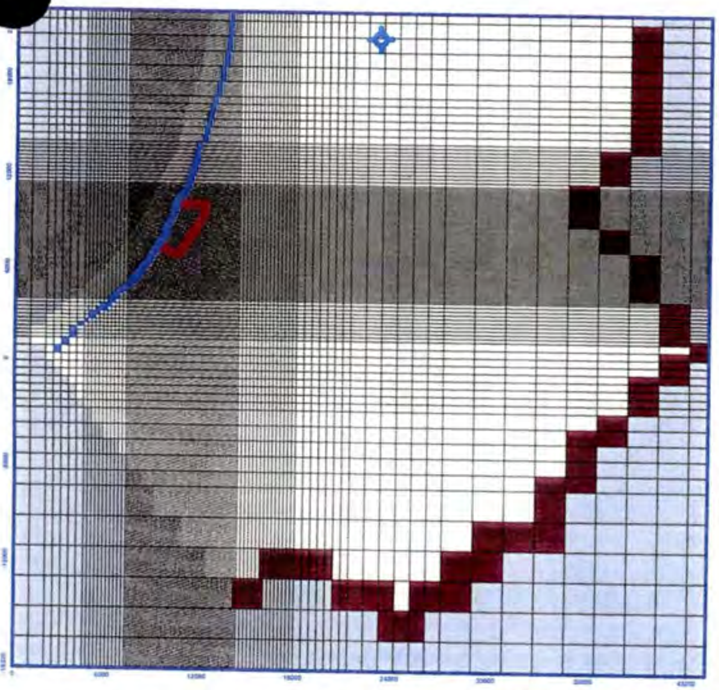
McDonald, M.G. and A. Harbaugh, 1988. *A Modular Three Dimensional Finite-Difference Groundwater Flow Model,* Techniques of Water Resources Investigations 06-A7, USGS.

Ritchey, J. D. and R.J. Schicht, 1982. "Ground-Water Management in the American Bottoms, Illinois", State, County, Regional, and Municipal Jurisdiction of Ground-Water Protection, Proceedings of the Sixth National Ground-Water Quality Symposium, Atlanta, Georgia, Sept. 14-22, 1982, National Water Well Association.

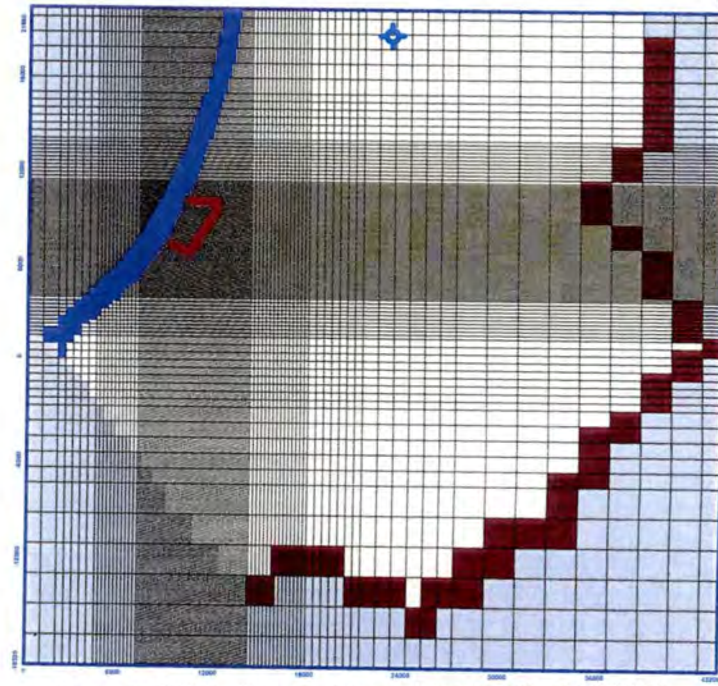
Schicht, R.J., 1965. *Ground-Water Development in East St. Louis Area, Illinois, Report of Investigation 51,* Illinois State Water Survey, Urbana, Illinois.

Schicht, R.J. and A.G. Buck, 1995. *Ground-Water Levels and Pumpage in the Metro-East Area, Illinois, 1986-1990,* Illinois State Water Survey, 1995.

Waterloo Hydrogeologic, No Date. *Visual MODFLOW User's Manual,* Waterloo, Hydrogeologic, Waterloo, Ontario.



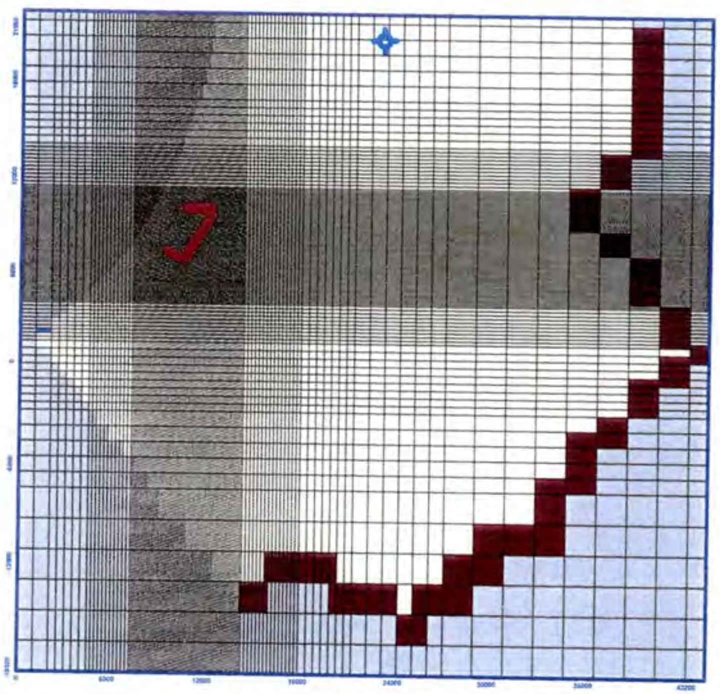
Layer 1: Shallow Horizon (approximately 0 - 26 ft BGS)



Layer 2: Middle Horizon (approximately 26 - 77 ft BGS)

N

PRELIMINARY



Layer 3: Deep Horizon (approximately > 77 ft BGS)

SCALE (ft)

0 6000 12,000

LEGEND

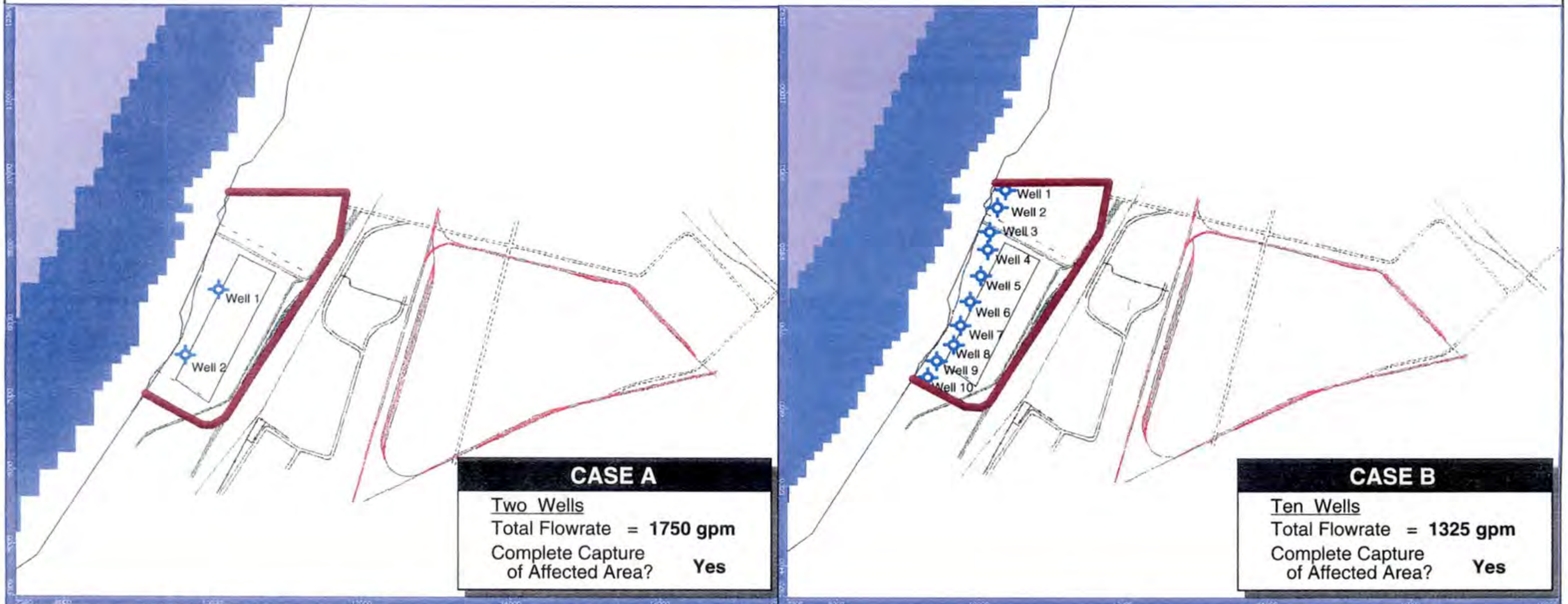
- Regional pumping center for highway dewatering
- River cells in MODFLOW model
- Approximate area of Site R affected groundwater
- Constant head cells
- Inactive cells

GROUNDWATER SERVICES, INC.

MODFLOW MODEL CONFIGURATION HYDRAULIC CONTAINMENT STUDY

Site R, W.G. Krummrich Plant, Sauget, Illinois
Solutia Inc.

GSI Job No:	G-2561	Drawn By:	CRW
Issued:	6/19/01	Chk'd By:	SKF
Revised:	———	Appv'd By:	CJN
Scale:	As Shown		**FIGURE 1**



N

LEGEND

- Pumping well
- River cells in MODFLOW model
- No detect line for SVOC in middle horizon constituents as indicated by Roux Associates, Inc. map dated 5/3/00.
- Inactive cells

PRELIMINARY

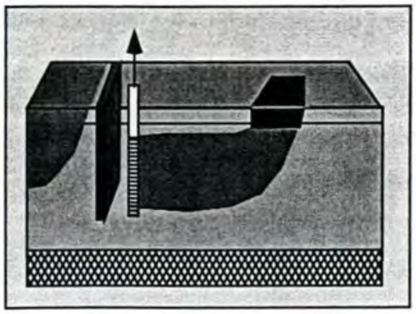
SCALE (ft)
0 1000 2000

GROUNDWATER SERVICES, INC.	GSI Job No. G-2561	Drawn By: CRW	**LOCATION OF PUMPING AND OBSERVATIONAL WELLS HYDRAULIC CONTAINMENT STUDY**
	Issued: 6/19/01	Chk'd By: SKF	Site R, W.G. Krummrich Plant, Sauget, Illinois
	Revised:	Aprv'd By: CJN	Solutia Inc.
	Scale: As Shown	**FIGURE 2**	

MASS CONTAINMENT STUDY

Site R
W.G. Krummrich Plant

Sauget, Illinois



PRELIMINARY

Submitted to
Solutia Inc.

June 20, 2001

Groundwater Services, Inc.
2211 Norfolk, Suite 1000, Houston, Texas 77098

EXECUTIVE SUMMARY

Site R is located in an area referred to as the American Bottoms on the east bank of the Mississippi River directly downgradient of the W.G. Krummrich Plant. The plume associated with Site R is approximately 2500 ft wide in a line perpendicular to groundwater flow. Higher concentrations are found in the shallow and middle horizon of the water-bearing unit with lower concentrations found in the deep horizon.

The objective of this study was to determine effective methods for controlling the discharge of dissolved constituent mass into the Mississippi River. A preliminary goal of achieving a 90% reduction in the organic mass flux to the river was established, and three general control alternatives were evaluated:

- Pumping wells alone;
- Pumping wells in combination with a fully-penetrating barrier wall;
- Pumping wells in combination with a partially-penetrating barrier wall.

A numerical groundwater flow model, MODFLOW, and a mass transport model, MT3D, were used to evaluate these alternatives (Figures 1-3).

Results

Two recovery wells located in high-concentration areas of the Site R groundwater plume are predicted to **capture greater than 90%** of the baseline organic mass discharge to the Mississippi River. The modeling analysis indicates that each well would need to be pumped at 300 gpm, resulting in a total flowrate of 600 gpm (Figure 4).

A ten-well recovery system spaced equidistant across the flowpath of affected groundwater from Site R appears to be **slightly less efficient** at capturing mass as some wells are not located in high concentration zones.

With a **partially penetrating barrier wall** downgradient of Site R, however, only **1 well pumping at 200 gpm** was predicted to capture 99% of the baseline mass discharge to the Mississippi River. The partially penetrating barrier wall used in the model was 3500 ft long and 77 ft deep. A fully penetrating barrier wall (109 ft deep) with a similar pumping scenario also resulted in capture of 99% of the baseline mass discharge.

INTRODUCTION

As requested by Solutia Inc. (Solutia), Groundwater Services, Inc. (GSI), has completed a study of mass containment options for affected groundwater associated with Site R near the W.G. Krummrich Plant in Sauget, Illinois. This report summarizes the approach and results of the study.

PROJECT BACKGROUND

Site R is located in an area referred to as the American Bottoms on the east bank of the Mississippi River directly downgradient of the W.G. Krummrich Plant. The geology of the area is described as consisting of unconsolidated valley fill deposits (Cahokia Alluvium) overlying glacial outwash material (Henry Formation). In general, the permeability of the unconsolidated material increases with depth, with the outwash material being comprised of medium- to coarse-grained sand and gravel. The hydrogeologic conceptual model divides the unconsolidated water-bearing unit into three horizons: the shallow horizon (generally 15-30 ft deep), the middle horizon (generally 30-70 ft deep), and the deep horizon (generally 70-110 ft deep). These unconsolidated deposits are underlain by limestone and dolomite bedrock.

Representative constituents associated with Site R include volatile organic constituents (VOCs) such as benzene, chlorobenzene, acetone, and 1,2-dichloroethane and semi-volatile organic constituents (SVOCs) such as phenol, 2-chloroaniline, and 2-nitrochlorobenzene. The plume associated with Site R is approximately 2500 ft wide in a line perpendicular to groundwater flow. Higher concentrations are found in the shallow and middle horizon, with lower concentrations found in the deep horizon.

The objective of this study was to determine effective methods for controlling the discharge of dissolved constituent mass into the Mississippi River. A preliminary goal of achieving a 90% reduction in the organic mass flux to the river was established, and three general control alternatives were evaluated:

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- Pumping wells in combination with a partially-penetrating barrier wall.

A numerical groundwater flow model, MODFLOW, and a mass transport model, MT3D, were used to evaluate these alternatives (Waterloo Hydrogeologic, no date).

MODEL DESCRIPTION

The MODFLOW groundwater flow model, developed by the U.S. Geological Survey (McDonald and Harbaugh, 1988) was used to simulate the movement of groundwater for baseline conditions and for various pumping scenarios. The MT3D mass transport model (Waterloo Hydrogeologic, no date). MODFLOW was used to evaluate the movement of dissolved constituents migrating in the groundwater.

Key MODFLOW Model Attributes, Assumptions, and Input Parameters

Key model attributes and assumptions for the MODFLOW model are listed below:

- A finite-difference grid, with 60 ft by 60 ft cells in the vicinity of Site R with cell size gradually increasing with distance from Site R, (Figure 1), was used for modeling the site.

- Three layers were used in the model: an unconfined shallow layer, a convertible confined/unconfined middle layer, and a confined deep layer. The top and bottom elevations of the water-bearing units were derived from available well logs and a cross-sectional map (Geraghty and Miller, date unknown, "Generalized Geologic Cross-Section A-A").

- Using data from the literature, slug test results, and calibration work, the following hydraulic conductivities were used in the model:

    - Shallow Horizon: $1x10^{-2}$ cm/sec
    - Middle Horizon: $1x10^{-1}$ cm/sec
    - Deep Horizon: $1x10^{-1}$ cm/sec

- Bedrock elevations, obtained by kriging data contained in Bergstrom and Walker (1956), were imported into the model.

- Geologic cross-section data developed by Bergstrom and Walker (1956) and USGS topographic maps were used to develop a simplified geometric vertical river boundary, where a rectangle was used to simulate the river from the western bank to the middle of the river, and a triangle was used to simulate the river from the mid-point to the eastern bank. The riverbed elevation for each river cell used in the model was derived using this simple geometric model.

- An average river level stage of 391 ft MSL was used for the river in the study area. River stage information was obtained from Schicht (1965) and Figure 10 of Schicht and Buck (1995).

- The riverbed conductance was assumed to be 3180 ft^2/day based on data developed by Schicht (1965).

- Constant head cells were used in the model to represent the eastern boundary of the modeled area (the bluff line). A constant elevation of 405 ft MSL was assigned to the constant head cells based on potentiometric surface information from November 1990 that was presented in Figure 14 of Schicht and Buck (1995).

- A surface infiltration rate of 7.8 inches per year was used in the model to represent infiltration from rainfall (Schicht, 1965).

- A regional pumping center of 4167 gpm was established in the model to represent ongoing highway dewatering projects in the East St. Louis area (Ritchey and Schicht, 1982).

- Wells used for evaluating plume capture were assumed to be screened only in the middle unit.

- Vertical barrier walls were assumed to have a hydraulic conductivity of $1x10^{-7}$ cm/sec and a thickness of 1 ft. A fully penetrating vertical barrier wall was assumed to be in place from the ground surface to the bedrock approximately 109 ft below ground surface. A partially penetrating vertical barrier wall was assumed to be in place from the ground surface to approximately 77 ft below ground surface.

Key MT3D Model Attributes, Assumptions, and Input Parameters

MT3D is a contaminant transport model that simulates the transport of dissolved constituents under the influence of advection (bulk groundwater flow), dispersion (spreading of constituent paths due to diffusion and preferential flowpaths), sorption (the adsorption of constituents to the aquifer media), and degradation (the destruction of constituents by chemical or biological processes). For this model:

- Adsorption and biodegradation were ignored in the simulations performed for this project to yield a conservative mass capture simulation. Dispersion was set a relatively low value to focus on this advection-dominated process and to minimize computational problems.

- Constant concentration sources were assumed to exist in the upper, middle, and deep aquifers (Figure 2). Source strengths were determined using the geometric mean of concentrations obtained within the highest concentration contours of the SVOC and VOC plume maps, respectively, developed by Roux Associates, Inc. (2000). A source strength of 2990 mg/L for SVOC and 27.5 mg/L for VOC was assigned to an area within the upper aquifer corresponding to the highest concentration contour of the SVOC figure. A source strength of 1524 mg/L for SVOC and 23.6 mg/L for VOC was assigned to an area within the middle aquifer corresponding to the highest concentration contour of the SVOC figure. A source strength of 18.6 mg/L for SVOC and 1.4 mg/L for VOC was assigned to an area within the deep aquifer corresponding to the highest concentration contour of the SVOC figure.

Modeling Approach

The MODFLOW model was run under steady-state conditions. Because the resulting potentiometric surfaces from the three layers were very similar, the potentiometric surface from the middle horizon was compared to the potentiometric surface reported for November 1990 reported by Schicht and Buck (1995). This comparison indicated that the general shape and values of the predicted potentiometric surface were similar to the reported potentiometric surface (including the cone of depression caused by the highway dewatering system). Therefore, the MODFLOW groundwater flow model was considered to yield a reasonable simulation of the aquifer system.

To establish representative starting concentrations, MT3D was run for 30 years, and the resulting concentrations caused by the source terms were compared to the concentrations observed in monitoring wells at the site. The source locations and strengths resulted in predicted concentrations that were within reasonable agreement with observed concentrations. This 30-year concentration distribution was then used as the initial condition for all subsequent mass transport modeling.

The project objective was to determine dissolved constituent mass discharged to the river for each alternative. For this calculation, the quantity of groundwater flowing into the river and the concentration of dissolved constituents in the groundwater discharged to the river was needed. These quantities were calculated using the ZoneBudget feature of MODFLOW in conjunction with mass transport simulations using MT3D.

ZoneBudget is a water balance component of the Visual MODFLOW package that reports the total quantity of groundwater flowing into the modeling domain from sources, and out of the domain through the model edges and internal sinks. ZoneBudget also reports the exchange of groundwater between adjacent zones set up by the user. To calculate the quantity of groundwater lost to the river, cells adjacent to the river were assigned as one zone, and the adjacent cells in the aquifer were assigned a different zone. Separate river and aquifer zones were established for each horizon since constituent concentrations differed between layers. The quantity of water flowing into the river zone from the aquifer zones was then reported by ZoneBudget.

Each horizon near Site R was divided into 3 zones. For each alternative, the mass lost to the river was calculated by the following procedure:

1. For each modeling scenario, MODFLOW, ZoneBudget and MT3D were run. The rate of groundwater discharge to the river from each aquifer zone reported by ZoneBudget was then used in the mass balance calculations.
2. The concentration in each aquifer zone that discharged to the river was estimated by placing a concentration observation well in each horizon zone. This concentration represented the dissolved constituent concentration discharged to the river from each zone. The concentrations were recorded by MT3D at periodic intervals for use in the mass balance calculations.
3. The total mass discharged to the river over the modeling period was calculated as the sum of the products of the river discharge and concentrations (after a five year simulation) in each zone as follows:

$$M_R = \sum_{1}^{\text{number of zones}} Q_i C_i$$

where Q_i = discharge rate of groundwater from zone i into the river
C_i = final constituent concentration in zone i
M_R = mass discharged to river

Modeling Results

Five different cases were evaluated:

- Case A: Baseline conditions (no pumping, no barrier wall);
- Case B: "Hot spot" pumping (2 pumping wells);
- Case C: Hydraulic containment (10 pumping wells between Site R and the river);
- Case D: Fully-penetrating barrier wall and pumping (wall + 1 pumping well);
- Case E: Partially-penetrating barrier wall and pumping (wall + 1 pumping well);

For **Case A,** a total of 6.8×10^5 kg/yr of SVOCs and VOCs was predicted by the model to discharge into the river from Site R (Figure 3a). This represents the baseline mass flux used for the mass containment analysis.

For **Case B**, the modeling analysis indicated that 2 "hot spot" wells pumping at 300 gpm each (total flowrate: 600 gpm) would capture 99% of the baseline mass discharge to the river (Figure 3b). At a flowrate of 100 gpm each (total flowrate 200 gpm), the two wells were predicted to capture 54% of the mass flux. In summary, for Case B:

	Number of Wells	Vertical Barrier Wall?	Total Pumping Rate from All Wells (gpm)	Percentage of Baseline Mass Discharge Captured by Wells (%)
Case B1	2	No	200	54 %
Case B2	2	No	600	91 %

For **Case C**, a ten-well hydraulic containment system was predicted to achieve the following mass removal rates (Figure 3c):

	Number of Wells	Vertical Barrier Wall?	Total Pumping Rate from All Wells (gpm)	Percentage of Baseline Mass Discharge Captured by Wells (%)
Case C1	10	No	500	72 %
Case C2	10	No	1300	99 %
Case C3	10	No	2200	100 %

For **Case D**, a fully penetrating vertical barrier wall in combination with a single well was predicted to achieve the following mass removal rate (Figure 3d):

	Number of Wells	Vertical Barrier Wall?	Total Pumping Rate from All Wells (gpm)	Percentage of Baseline Mass Discharge Captured by Wells (%)
Case D	1	Fully Pen.	200	99 %

Note: Slurry wall is 3500 long, 109 ft deep.

For **Case E**, a partially penetrating vertical barrier wall in combination with a single well was predicted to achieve the following mass removal rate (Figure 3e):

	Number of Wells	Vertical Barrier Wall?	Total Pumping Rate from All Wells (gpm)	Percentage of Baseline Mass Discharge Captured by Wells (%)
Case E	1	Partially Penetrating	200	99 %

Note: Slurry wall is 3500 long, 77 ft deep.

A summary of all the modeling results is provided below and in Table 1 and Figure 4:

	Number of Wells	Vertical Barrier Wall?	Total Pumping Rate from All Wells (gpm)	Percentage of Baseline Mass Discharge Captured by Wells (%)
Case A	0	No	None	0%
Case B1	2	No	200	54 %
Case B2	2	No	600	91 %
Case C1	10	No	500	72 %
Case C2	10	No	1300	99 %
Case C3	10	No	2200	100 %
Case D	1	Fully Pen.	200	99 %
Case E	1	Partially Penetrating	200	99 %

To verify the adequacy of using three zones per horizon to predict mass discharge to the river, each horizon was more finely divided into 10 zones, an observation well was placed in each zone, and one case (Case E) was rerun. The finely divided run gave almost the same results as the original run, indicating that the original system (three observation wells in each horizon) provided an accurate representation of mass flux to the river.

KEY POINT: PUMPING RATE REQUIRED FOR MASS CAPTURE

Two recovery wells located in high-concentration areas of the Site R groundwater plume are predicted to **capture greater than 90%** of the baseline organic mass discharge to the Mississippi River. The modeling analysis indicates that each well would need to be pumped at 300 gpm, resulting in a total flowrate of 600 gpm (Figure 4).

A ten-well recovery system spaced equidistant across the flowpath of affected groundwater from Site R appears to be **slightly less efficient** at capturing mass as some wells are not located in high concentration zones.

With a **partially penetrating barrier wall** downgradient of Site R, however, only **1 well pumping at 200 gpm** was predicted to capture 99% of the baseline mass discharge to the Mississippi River. The partially penetrating barrier wall used in the model was 3500 ft long and 77 ft deep. A fully penetrating barrier wall (109 ft deep) with a similar pumping scenario also resulted in capture of 99% of the baseline mass discharge.

REFERENCES

Bergstrom, R.E. and T.R. Walker, 1956. *Groundwater Geology of the East St. Louis Area, Illinois*, Illinois State Geological Survey, Urbana, Illinois.

McDonald, M.G. and A. Harbaugh, 1988. *A Modular Three Dimensional Finite-Difference Groundwater Flow Model*, Techniques of Water Resources Investigations 06-A7, USGS.

Ritchey, J. D. and R.J. Schicht, 1982. "Ground-Water Management in the American Bottoms, Illinois," State, County, Regional, and Municipal Jurisdiction of Ground-Water Protection, Proceedings of the Sixth National Ground-Water Quality Symposium, Atlanta, Georgia, Sept. 14-22, 1982, National Water Well Association.

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Waterloo Hydrogeologic, No Date. *Visual MODFLOW User's Manual*, Waterloo Hydrogeologic, Waterloo, Ontario.

GROUNDWATER SERVICES, INC.

TABLE 1
SUMMARY OF MODELING RESULTS

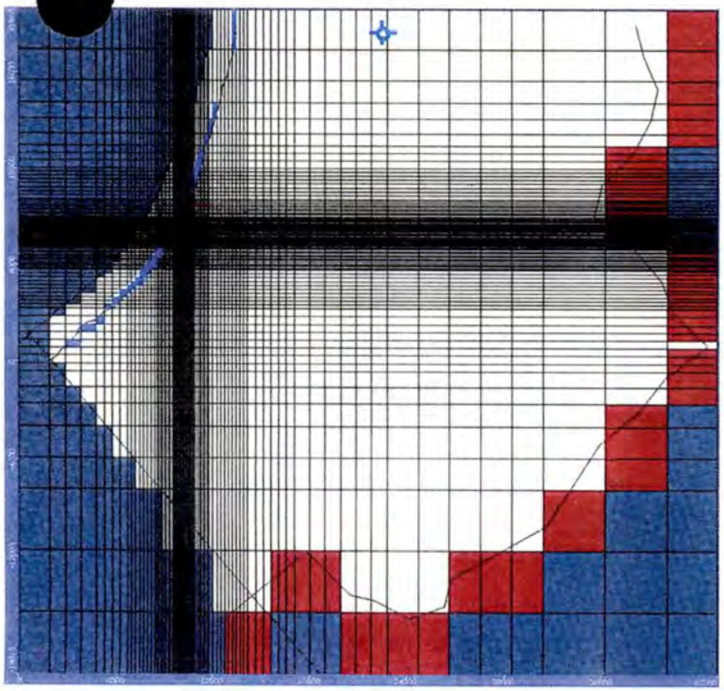
W. G. Krummrich Plant, Sauget, Illinois
Solutia. Inc.

Observation Well	Layer	Flow Through Zone (gpm)							
		A	B1	B2	C1	C2	C3	D	E
1	1	4.78E+01	4.69E+01	4.50E+01	4.55E+01	4.20E+01	3.82E+01	4.46E+01	4.46E+01
2	1	7.70E+01	7.56E+01	7.28E+01	7.30E+01	6.69E+01	6.02E+01	6.98E+01	6.98E+01
3	1	5.43E+01	5.29E+01	5.01E+01	5.16E+01	4.75E+01	4.30E+01	4.90E+01	4.90E+01
4	2	6.07E+01	4.45E+01	1.96E+01	2.48E+01	0.00E+00	0.00E+00	1.59E+01	1.59E+01
5	2	1.05E+02	8.83E+01	5.42E+01	4.93E+01	4.78E+00	7.35E+00	1.39E+01	1.39E+01
6	2	1.05E+02	7.82E+01	4.04E+01	5.92E+01	3.29E+01	3.00E+01	4.38E+01	4.38E+01
7	3	1.14E+01	4.85E+00	4.52E-01	1.43E-02	0.00E+00	0.00E+00	1.59E+01	0.00E+00
8	3	2.77E+01	1.65E+01	3.44E+00	3.04E+00	2.31E+00	3.18E+00	1.39E+01	2.61E+00
9	3	2.93E+01	1.62E+01	6.99E+00	1.02E+01	8.01E+00	7.49E+00	4.38E+01	1.27E+01
Total		5.18E+02	4.24E+02	2.93E+02	3.17E+02	2.04E+02	1.89E+02	3.11E+02	2.52E+02

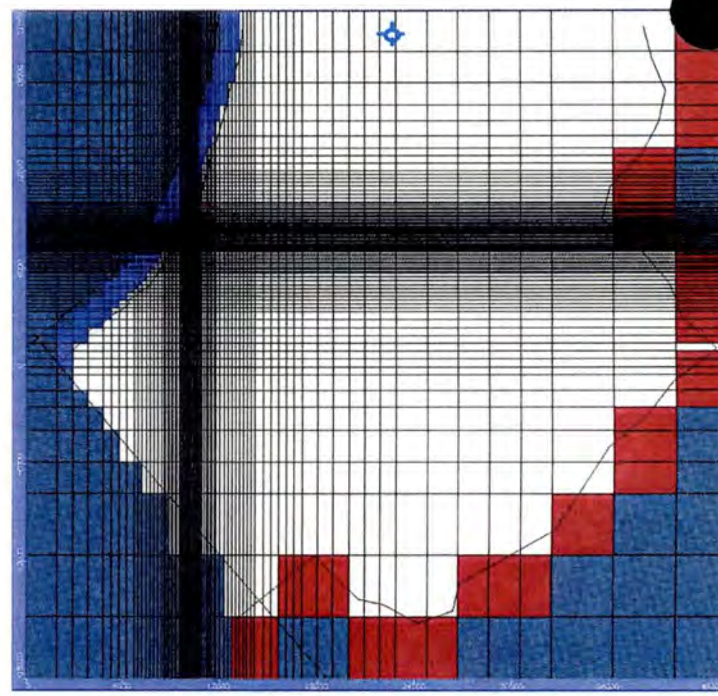
Observation Well	Layer	Concentration (mg/L)							
		A	B1	B2	C1	C2	C3	D	E
1	1	1.05E+03	3.74E+02	2.83E+01	2.08E+02	7.48E+00	2.87E+00	2.45E+01	2.45E+01
2	1	4.55E+02	3.47E+02	2.08E+02	2.01E+02	8.56E+00	4.60E+00	3.44E+01	3.44E+01
3	1	1.16E+03	5.58E+02	4.45E+01	5.51E+02	4.20E+01	2.64E+00	6.37E+00	6.37E+00
4	2	8.14E+02	2.52E+02	1.25E+01	1.44E+02	4.16E+00	2.12E+00	1.08E+01	1.08E+01
5	2	1.57E+02	1.00E+02	3.01E+01	2.18E+01	3.78E+00	2.49E+00	4.76E+00	4.76E+00
6	2	1.19E+03	8.06E+02	2.76E+02	6.11E+02	2.50E+00	3.18E-07	3.71E+00	3.71E+00
7	3	2.00E+01	2.00E+01	2.00E+01	2.00E+01	2.00E+01	2.00E+01	1.90E+01	1.90E+01
8	3	2.00E+01	2.00E+01	2.00E+01	2.00E+01	2.00E+01	2.00E+01	1.87E+01	1.87E+01
9	3	6.93E-05	6.08E-05	4.30E-01	5.68E-05	6.57E-07	2.24E-08	1.53E+00	1.53E+00
Total		4.87E+03	2.48E+03	6.39E+02	1.78E+03	1.09E+02	5.48E+01	1.24E+02	1.24E+02

Observation Well	Layer	Mass (kg/yr)							
		A	B1	B2	C1	C2	C3	D	E
1	1	9.96E+04	3.49E+04	2.53E+03	1.88E+04	6.25E+02	2.19E+02	2.17E+03	2.17E+03
2	1	6.97E+04	5.22E+04	3.01E+04	2.91E+04	1.14E+03	5.51E+02	4.78E+03	4.78E+03
3	1	1.26E+05	5.87E+04	4.44E+03	5.66E+04	3.97E+03	2.26E+02	6.22E+02	6.22E+02
4	2	9.83E+04	2.22E+04	4.87E+02	7.12E+03	0.00E+00	0.00E+00	3.43E+02	3.43E+02
5	2	3.30E+04	1.76E+04	3.25E+03	2.14E+03	3.60E+01	3.65E+01	1.32E+02	1.32E+02
6	2	2.48E+05	1.25E+05	2.22E+04	7.20E+04	1.63E+02	1.90E-05	3.23E+02	3.23E+02
7	3	4.54E+02	1.93E+02	1.80E+01	5.70E-01	0.00E+00	0.00E+00	6.02E+02	0.00E+00
8	3	1.11E+03	6.57E+02	1.37E+02	1.21E+02	9.21E+01	1.27E+02	5.17E+02	9.69E+01
9	3	4.05E-03	1.96E-03	5.99E+00	1.15E-03	1.05E-05	3.34E-07	1.33E+02	3.85E+01
Total		6.76E+05	3.12E+05	6.31E+04	1.86E+05	6.03E+03	1.16E+03	9.62E+03	8.51E+03
Percent Capture			54%	91%	72%	99%	99.8%	99%	99%

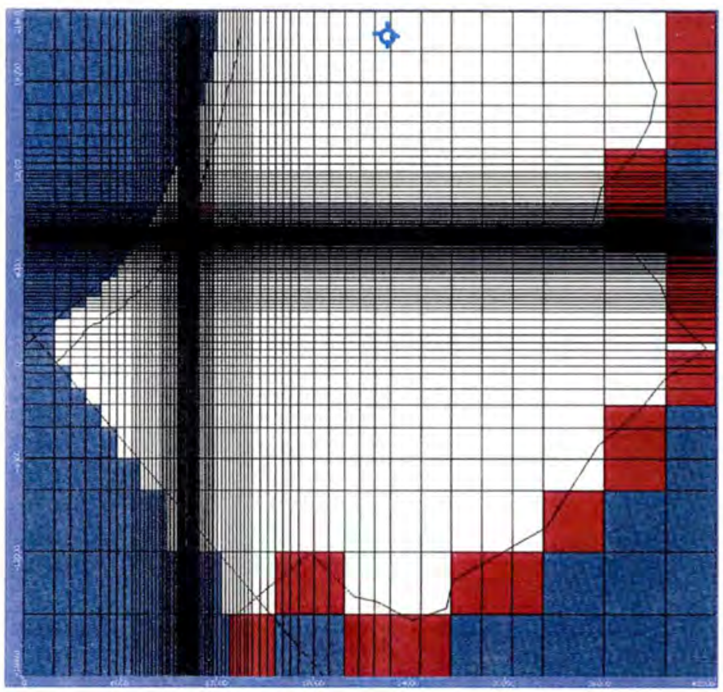
Note:
A = Baseline conditions (no pumping, no barrier wall).
B1 = "Hot spot" pumping (2 pumping wells at 200 gpm total pumping rate).
B2 = "Hot spot" pumping (2 pumping wells at 600 gpm total pumping rate).
C1 = Hydraulic containment (10 pumping wells between Site R and the river, 500 gpm total pumping rate).
C2 = Hydraulic containment (10 pumping wells between Site R and the river, 1300 gpm total pumping rate).
C3 = Hydraulic containment (10 pumping wells between Site R and the river, 2200 gpm total pumping rate).
D = Fully-penetrating barrier wall and pumping (wall + 1 pumping well at 200 gpm).
E = Partially-penetrating barrier wall and pumping (wall + 1 pumping well at 200 gpm).



Layer 1: Shallow Horizon (approximately 0 - 26 ft BGS)



Layer 2: Middle Horizon (approximately 26 - 77 ft BGS)



Layer 3: Deep Horizon (approximately > 77 ft BGS)

N

PRELIMINARY

LEGEND

- Regional pumping center for highway dewatering
- River cells in MODFLOW model
- Approximate area of Site R affected groundwater
- Constant head cells
- Inactive cells

SCALE (ft)

0 6000 12,000

GROUNDWATER SERVICES, INC.

MODFLOW MODEL CONFIGURATION
MASS REMOVAL STUDY

Site R, W.G. Krummrich Plant, Sauget, Illinois
Solutia Inc.

GSI Job No: G-2561		Drawn By: CRW	
Issued: 6/19/01		Chk'd By: SKF	
Revised: ______		Appv'd By:	
Scale: As Shown		**FIGURE 1**	

Layer 1: Shallow Horizon (approximately 0 - 26 ft BGS)

Layer 2: Middle Horizon (approximately 26 - 77 ft BGS)

N

Layer 3: Deep Horizon (approximately > 77 ft BGS)

LEGEND

- River cells in MODFLOW model
- No detect line for SVOC in middle horizon constituents as indicated by Roux Associates, Inc. map dated 5/3/00.
- Inactive cells
- Constant concentration source in Layer 1 (SVOC = 2990 mg/L; VOC = 27.5 mg/L)
- Constant concentration source in Layer 2 (SVOC = 1524 mg/L; VOC = 23.6 mg/L)
- Constant concentration source in Layer 3 (SVOC = 18.6 mg/L; VOC = 1.4 mg/L)
- 0.1 — Equal concentration line SVOC and VOC, mg/L

PRELIMINARY

SCALE (ft)

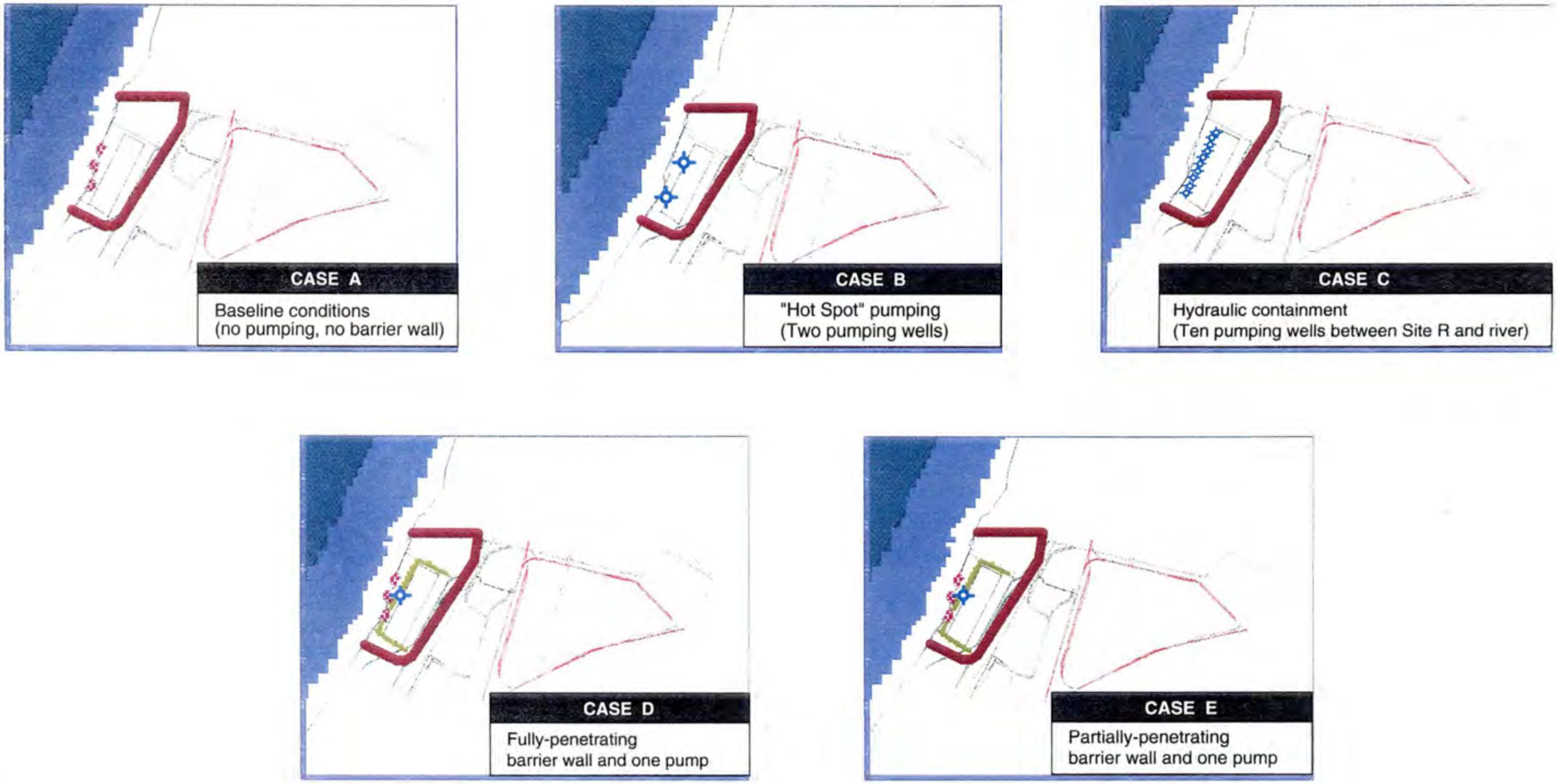
0 1200 2600

GROUNDWATER SERVICES, INC.

GSI Job No.	G-2561	Drawn By:	CRW
Issued:	6/19/01	Chk'd By:	SKF
Revised:	————	Aprv'd By:	CJN
Scale:	As Shown		**FIGURE 2**

INITIAL MODEL CONCENTRATION MASS REMOVAL STUDY

Site R, W.G. Krummrich Plant, Sauget, Illinois
Solutia Inc.



N

LEGEND

- Pumping well
- Observation well
- River cells in MODFLOW model
- No detect line for SVOC in middle horizon constituents as indicated by Roux Associates, Inc. map dated 5/3/00.
- Inactive cells
- Barrier wall

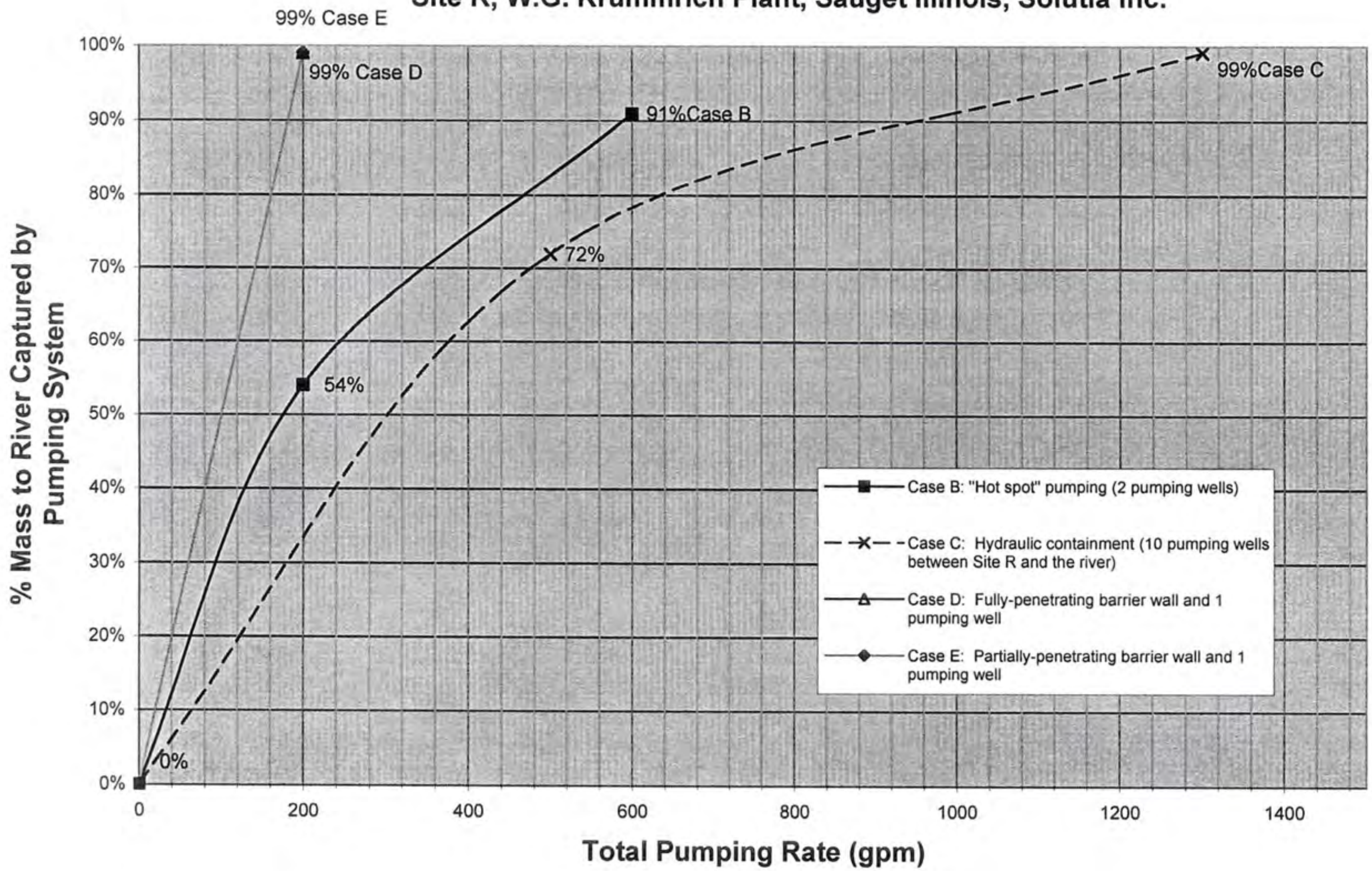
PRELIMINARY

SCALE (ft)
0 2000 4000

GROUNDWATER SERVICES, INC.	GSI Job No. G-2561	Drawn By: CRW	**LOCATION OF PUMPING WELLS, OBSERVATION WELLS, AND BARRIER WALLS, CASES A- E MASS REMOVAL STUDY**
	Issued: 6/19/01	Chk'd By: SKF	Site R, W.G. Krummrich Plant, Sauget, Illinois
	Revised:	Aprv'd By: CJN	Solutia Inc.
	Scale: As Shown	**FIGURE 3**	

PRELIMINARY
GSI Job No. G-2561
Issued: 5/20/01

Figure 4
PERCENTAGE MASS CAPTURE
(After Five Years, Avg. River Level)
Site R, W.G. Krummrich Plant, Sauget Illinois, Solutia Inc.



SOLUTIA - 056

July 30, 2001

DE-9J

Mr. Robert Hiller
Solutia Inc.
500 Monsanto Avenue
Sauget, IL 62206-1198

RE: W.G. Krummrich Plant
Containment Studies
Solutia Inc.
ILD 000 802 702

Dear Mr. Hiller:

At our June 21, 2001 meeting held at the U.S. EPA, Region 5 office, you provided the *Hydraulic Containment Study* and *Mass Containment Study* (dated June 20, 2001) for Site R at the W.G. Krummrich Plant.

The studies investigate how contaminants in the groundwater plume could be contained and prevented from discharging to the Mississippi River. The studies concluded that a partially penetrating barrier wall combined with a well pumping at 200 gpm would capture 99% of the mass contaminant discharge to the Mississippi River.

U.S. EPA has reviewed the studies and our comments are enclosed. If you have any questions, I can be reached at (312) 886-7566 or at bardo.kenneth@epa.gov

Sincerely yours,

Kenneth S. Bardo

Kenneth S. Bardo
EPA Project Manager
Corrective Action Section

Enclosure

cc: Jim Moore, IEPA
Gina Search, IEPA

bcc: Michael McAteer, Superfund
Rick Hersemann, Tetra Tech EMI

OFFICIAL FILE COPY

ENCLOSURE

A. HYDRAULIC CONTAINMENT STUDY

General Comments

1. The report does not provide all the details necessary to evaluate the accuracy of the groundwater model. All assumptions, hydraulic properties, and results of the numerical model should be clearly discussed in the text. For example:

- Conditions in the modeled domain that require the use of "convertible" nodes to the model confined/unconfined middle layer are not discussed.

- Leakance values for confining layers are not provided.

- Bedrock elevations were imported into the model; however, elevations are not mentioned in the report. Factors and parameters used in the model should be discussed in the report.

2. The report states in several places that a transient simulation was not performed due to the steady-state simulation being modeled on a "low river level scenario." While this approach may accurately reflect the most difficult hydraulic capture scenario, a transient model should be developed to allow for thorough model calibration.

3. The report does not clearly discuss model calibration and does not mention if a sensitivity analysis was performed on the model.

Specific Comments

1. **Executive Summary:** The plume is only partially associated with Site R. The plume discharging to the Mississippi River may have many sources and groundwater monitoring data shows that it originates at the active manufacturing portion of the W.G. Krummrich Plant, approximately 6000' east of the Mississippi River.

2. **Results, Page 1, Last Paragraph:** This paragraph states that the effects of changing Mississippi River water levels were not considered in the analysis. A full understanding of transient conditions associated with seasonal fluctuations in river levels, aquifer saturated thickness, and seasonal

groundwater withdrawals requires the development of a transient groundwater model. A transient model should be produced and calibrated using historic groundwater data.

3. **Key Model Attributes, Assumptions, and Input Parameters, Page 3:** The first bullet states that the model contains a finite-difference grid, with 120- by 120-foot cells around Site R, and with cell size gradually increasing outward from Site R. The bullet does not state the method by which cell sizes were selected moving out from Site R, nor does the bullet indicate a range of cell sizes. Typically, a value of 1.5 is used as a factor by which to increase cell size with each successive change in cell dimension. The bullet should indicate the method used to select cell sizes.

4. **Key Model Attributes, Assumptions, and Input Parameters, Page 3:** The second bullet indicates that a "convertible" confined/unconfined middle layer was used in the model. It is assumed that "convertible" indicates that the cells can toggle between confined and unconfined nodes, depending on hydraulic conditions in the modeled domain. However, the model does not specify if conditions in the modeled domain require the cells to convert from confined to unconfined. The bullet should explain if the cells changed during the course of the simulation and under what conditions.

5. **Key Model Attributes, Assumptions, and Input Parameters, Page 3:** The fourth bullet states that bedrock elevations were imported into the model; however, the bullet does not specify the elevations. The bullet should present the bedrock elevations used in the model.

6. **Key Model Attributes, Assumptions, and Input Parameters, Page 4:** The fourth bullet states that a regional pumping center was included in the groundwater model. The pumping center discharges at a rate of 4,167 gallons per minute. The report does not specify the intervals the pumping center is screened in. Based on Figure 1, it would appear that the pumping center affects all three modeled saturated zones; however, this is unclear. The bullet also does not address the radius of influence of the pumping well. The well is located about 1,000 feet below the northern boundary of the modeled domain. The report does not indicate if the cone of depression from the pumping center intersects the northern boundary of the model, or if it affects contaminant plume migration. While the pumping center appears to be located about 3 miles from the Site R contaminant plume and may be too distant to influence contaminant migration, the report should mention the hydraulic impact of this pumping center.

7. **Modeling Approach, Page 4, First Paragraph:** This paragraph appears to discuss calibrating the groundwater model. The paragraph mentions only that the potentiometric surface produced from the groundwater model was compared to a historical data set. The report does not mention if a statistical analysis of the data was performed to compare the calculated data to the observed historical data set. The report also does not indicate if calibration was performed by a trial-and-error method or if an automated calibration was performed. The report should discuss if a statistical analysis, such as root-mean-squared error calculation, was performed to determine the deviation of the calculated data from the historical data.

B. MASS CONTAINMENT STUDY

General Comment

The general and specific comments on the "Hydraulic Containment Study" are applicable and should be addressed in the "Mass Containment Study."

Specific Comments

1. **Executive Summary, Page 1, Second Paragraph:** The report states that a preliminary goal was set at achieving 90 percent reduction in organic mass flux to the river. However, the report does not provide a rationale for setting this goal such as to meet Federal maximum contaminant levels and Illinois water quality standards.

   Solutia must ultimately demonstrate that any contaminants discharging to the Mississippi River after implementation of the containment remedy do not significantly impact the river. For example, modeling predicts a total of 680,000 kg/year of SVOCs and VOCs being discharged to the river. Even a 99% capture rate would result in 6,800 kg/year of SVOCs and VOCs being discharged to the river, which could still be a significant amount.

2. **Key MODFLOW Model Attributes, Assumptions, and Input Parameters, Page 3:** The first bullet states that the model contains a finite-difference grid, with 60- by 60-foot cells around Site R, and with cell size gradually increasing outward from Site R. The grid size specified in this statement does not agree with the grid size specified in the "Hydraulic Containment Study," which states that a cell size of 120- by 120-feet was used around Site R. The report should discuss why the cell size differs from the "Hydraulic Containment Study" or indicate the accurate cell size.

3. **Key MODFLOW Model Attributes, Assumptions, and Input Parameters, Page 4:** The first bullet states that an average river level stage of 391 feet above mean sea level (amsl) was used for the river stage in the study area. The "Hydraulic Containment Study" model used a "low river level" of 380.9 feet amsl. The report does not indicate the reason for the difference in river stages used between the two simulations. This difference may significantly affect the estimate of discharge necessary from the series of pumping wells used to capture dissolved contamination. The report should discuss why the river stages differ in the two containment studies or indicate the accurate river stage.

4. **Key MODFLOW Model Attributes, Assumptions, and Input Parameters, Page 4:** The second bullet states that a riverbed conductance of 3,180 square feet per day (ft^2/day) was used for the mass containment study simulation. This value of riverbed conductance differs from the value of 3,182 ft^2/day given in the "Hydraulic Containment Study". While this represents a small difference for each cell, over the entire length of the river, this discrepancy could affect the estimate of discharge necessary from the series of pumping wells used to capture dissolved contamination. The report should either include a discussion of why the riverbed conductance value differs in the two containment studies or indicate the accurate riverbed conductance.

5. **Modeling Approach, Page 5, Second Paragraph:** The paragraph discusses how initial conditions were determined for the concentration distribution for mass transport modeling. The text states that the MT3D simulation spanned a time period of 30 years, and the resulting contaminant concentrations were compared to concentrations observed in monitoring wells at the site. However, the text does not provide a criterion for determining whether the observed data compared favorably to calculated data. The text should discuss (1) the criterion used to examine the fit of observed to calculated data and (2) a discussion of any statistical analyses applied to the data.

SOLUTIA - 057

Kenneth Bardo/R5/USEPA/US

08/01/2001 04:30 PM

To gina.search@epa.state.il.us

cc

bcc

Subject Solutia

Gina - I e-mailed the attached comments on the MCB RAP to Bob Hiller on July 31. He had sent the Compliance Commitment Agreement dated June 7, 2001 a few days ago. I requested further discussions. If you have any questions, give me a call. - Ken

Solutia MCB Spill Comments.w

July 31, 2001

U.S. EPA Comments on MCB Recovery Testing Results and Remedial Action Plan for the Solutia W.G. Krummrich Plant Dated 5/21/01

- The June 6, 2001 cover letter states that a diaphragm pump was installed and activated on May 31, 2001 and that data will be collected. Provide the volume of free-product recovered since installation of the diaphragm pump. Provide the data collected to determine the pumping cycle and the effectiveness of adding additional wells.

- Two additional recovery wells (RW-2 and RW-3) were to be evaluated after review of 2 weeks of recovery process operating data. Provide the status of the two possible additional recovery wells.

- Provide the basis for defining the effective removal of free product (>10% free product vs. groundwater). The extent of product recovery should consider the volume of MCB spilled (approx. 6,700 gallons or 58,000 pounds).

- Provide a facility map that depicts the location of the MCB spill relative to other solid waste management units and areas of concern.

- Provide any written or verbal responses from IEPA regarding the Compliance Commitment Agreement dated June 7, 2001.

- Solutia proposes that further remedial action in the MCB spill area, beyond the recovery of free-product from the upper 15-feet of silty soil, be included in corrective action pursuant to the Consent Order issued by U.S. EPA.

  Under the Consent Order, applicable cleanup goals are risk-based. Soil remediation for industrial property requires cleanup to risk-based levels based on potentially complete exposure pathways and protection of groundwater. Applicable risk-based concentrations (TACO) for detected concentrations of chlorobenzene, benzene, and dichlorobenzenes that need to be met could be based on soil-to-groundwater leaching, human exposure from incidental ingestion and dermal contact, and human exposure from inhalation of ambient air and within buildings. The ultimate soil cleanup goal is the lowest soil concentration provided for in TACO, based on these potentially complete exposure pathways.

- The Consent Order provides for Solutia to remediate areas as early as possible to meet the environmental indicator requirements. Remediation of soil in the MCB process area can be initiated both to help meet the environmental indicator requirements and determine the feasible technology for addressing all soil contaminated at the facility with similar benzene-ring compounds.

- Removal of as much free-product as technically feasible will assist in meeting the required soil cleanup goals. Following the removal of all free-product as is technically feasible, U.S. EPA anticipates that additional remediation will then commence in the MCB spill area to achieve the soil cleanup goals.

SOLUTIA - 060

Hanyer

SOLUTIA

Applied Chemistry, Creative Solutions

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Brent J. Gilhousen
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E-Mail: BJGILH@Solutia.com

October 31, 2001

Mr. Robert Springer
Director, Waste, Pesticides and Toxics Division
D-8J
U.S. Environmental Protection Agency, Region V
77 W. Jackson Boulevard
Chicago, IL 60604-3590

Mr. William Muno
U.S. Environmental Protection Agency, Region V
S-6J
77 West Jackson Boulevard
Chicago, IL 60604-3590

Re: Sauget, Illinois Sites

Dear Bob and Bill,

I thought it would be useful at this time to send you a follow-up letter to our meeting in Chicago on October 3, 2001, at which we discussed matters regarding the Administrative Order on Consent entered into between EPA and Solutia on May 3, 2000 under RCRA with respect to the Krummrich facility as well as the status of activities with respect to the Superfund proceedings concerning Sauget Area 1 and Sauget Area 2. The purpose of this letter is to confirm some of the points we discussed during the meeting and to report on a few significant developments subsequent to it.

First, however, I want to express to both of you our great appreciation for the meeting itself. I and my colleagues who participated in the meeting on behalf of Solutia all felt that it was extremely productive. We were particularly grateful for the opportunity to meet with both of you

and your key staff all together, to be able to discuss both RCRA and Superfund elements of the situation in a coordinated manner. We appreciated your setting aside two hours without interruption for the meeting and also appreciated that members of your staff were able to continue further discussion after that. All of this made it possible to have a comprehensive review of all the salient topics and a useful exchange of information and views on the critical issues. Thank you for giving your personal attention to these matters.

Background

As you know well and as we explained during our presentation, there is a long and complex history regarding the Sauget region, both in terms of its industrial operations and in terms of attention by EPA (and the State of Illinois) to hazardous waste contaminants found there. I will not recite that full history here, but will mention a few points of particular importance and relevance.

The Sauget "region" may be regarded for these purposes as an area of approximately one to two miles wide (east to west) and roughly three miles long (north to south) located along the Mississippi River in southern Illinois across from St. Louis. It has been industrialized throughout the past 100 years, though in the more recent past industrial use has diminished, leaving the area economically weakened. Throughout most of that history Monsanto was the largest employer in the area. As a result of Monsanto's creation of an independent publicly traded corporation in 1997 (Solutia Inc.), placing certain assets within the new corporation and distributing the stock of that corporation ("spin off") to the stockholders of Monsanto, Solutia now owns and conducts the remaining operations of the Krummrich facility. Furthermore, Solutia has assumed the legal responsibilities related to the former Monsanto operations from processes at that facility. A large number of other companies also operate, or in the past have operated, industrial facilities in this area. One result of this industrial history has been a considerable number of hazardous waste disposal units and contamination of the soils and groundwater.

In 1996 EPA proposed to list a portion of this area as a Superfund site on the National Priority List. The area, encompassing several specific hazardous waste disposal units, is now known as "Sauget Area 1." In 1995 EPA undertook a removal action at one of the units (Site "G"). In 1998 EPA sent special notice letters to 26 parties it considered potentially responsible parties, requesting that they agree to perform an Engineering Evaluation/Cost Analysis (EE/CA) for the specified source areas and an RI/FS for the groundwater. Only Solutia was willing to undertake that work, and on January 21, 1999 Solutia (and Monsanto) and EPA entered into an AOC covering those commitments. A cost recovery suit was initiated by the federal government in early 1999 and expanded in 2000 by Solutia -- these claims against a large number of defendants are currently in the discovery phase of litigation. Meanwhile, Solutia has gone forward on its own at substantial cost to conduct cleanups at the source areas, achieving major progress toward cleanup of the area and essentially eliminating any risk from direct exposure to contaminated soils within Area 1. Solutia has also completed and submitted to EPA a draft RI/FS for Area 1 groundwater.

EPA later identified additional portions of the region (now known as Sauget Area 2) as subject to a second set of Superfund proceedings. The locations covered by Area 2 generally lie to the west of the locations covered by Area 1, placing them closer to the river and generally downgradient from Area 1 in terms of groundwater migration. In the summer of 2000 EPA issued special notice letters to nearly 100 PRPs, calling on them to assume responsibility for the sites. A group of approximately 16 of those parties (known as "SA2SG") agreed with EPA pursuant to an AOC signed in November 2000 to perform an RI/FS for Area 2 and currently is negotiating a Site Sampling Plan.

The former Monsanto facility known as the Krummrich plant lies just to the north of the Area 1 locations and like them is set back roughly a mile from the river. Solutia continues to conduct certain industrial operations at that plant, though with a markedly reduced work force. This facility is subject to regulation under RCRA, and on May 3, 2000 Solutia and EPA signed an AOC under RCRA regarding corrective action. Solutia has conducted extensive sampling and analysis under that AOC, focused particularly on contaminants found at the edge of the river downgradient from the plant. Pursuant to the AOC, Solutia has submitted two technical reports to EPA reflecting the results of that work, a Description of Current Conditions and an Ecological Risk Assessment.

Groundwater Contamination

One fact of major importance has emerged from the field investigation work carried out to prepare the draft RI/FS for Area 1 and the technical reports for the AOC under RCRA. It is that the groundwater plumes from all three areas converge and are commingled. The RI/FS for Area 1 indicates that groundwater contamination from that area, specifically from Site I, flows westward directly toward Sites R and Q of Area 2 which lie very close to the riverfront, though it appears that this plume does not actually reach the river itself. Similarly, the data collected with respect to the plume of contaminated groundwater from the Krummrich facility also flows westward toward the river and also reaches Site R though that plume also apparently stops before reaching the river itself (see discussion below).

This convergence and commingling of plumes from Area 1, Area 2, and Krummrich create a situation in which the development of remedial action proposals for each of the three areas are inextricably interwoven. It would be difficult, if not impossible, to develop a sound, environmentally protective and cost-effective groundwater control program for these three areas by addressing any of them in isolation from the others. The nature and complexity of the groundwater issues presented require an integrated, comprehensive approach. During our meeting, we urged EPA to adopt such an approach. We appreciated your receptivity to that

consideration, and we understand that you intend to undertake necessary steps to assure that the needed coordination is achieved.

Specific Suggestions

During our meeting we presented three specific recommendations with respect to the Sauget region, as follows:

1. As stated above, we urged that an integrated approach to the management of the three areas be adopted that would harmonize the RCRA and Superfund regulatory requirements and would synthesize the analysis regarding the control of contaminated groundwater. Also as stated above, we believe that you are in agreement with this as a matter of principal.

2. As one step to achieve the needed integration, we proposed that EPA not proceed to make a final remedy selection or develop a ROD for Area 1 on the basis on the RI/FS for that area but defer that until the RI/FS for Area 2 is completed. That would also mesh with the schedule for completion of the Corrective Measures Study for Krummrich. Our impression from your responses during the meeting and some subsequent contacts with staff is that you are inclined to regard that as a sensible approach, but we look forward to further communication of your position on this point when you view that as timely. We also should note that this approach would not preclude moving ahead with certain interim measures if upon further analysis it appears that any such actions are required and can be planned without risk of being inconsistent with the final remedy selection.

3. We also suggested that to enhance coordination a single consolidated report should be developed for submission to EPA to satisfy the requirements for an RI/FS at Area 2 and for a corrective measures study (CMS) for Krummrich. We understood you to express preliminary approval of that concept, subject to the need to explore further the feasibility and mechanics of doing so. In addition to that analytical work, we would also point out that it would be necessary to obtain the agreement of other participants in the SA2SG that this would be desirable. We would like to probe the details of this with you somewhat deeper and then if this does indeed appear to be a promising approach we would want to discuss it within the SA2SG to obtain their concurrence.

Compliance with EI at Krummrich

Another topic we discussed concerned requirements in the AOC for Krummrich that Solutia demonstrate by January 1, 2002 compliance with the Environmental Indicator for control of migration of contaminated groundwater. We explained our position that modeling analysis would currently show that this plume has been stabilized and that contaminants from Krummrich itself are not actually reaching the riverfront. You responded that in your view we would not be able at this time to demonstrate such compliance and could not properly rely on modeling alone to do so. You suggested that we submit a request for extension of the AOC deadline, and within the next few weeks we do intend to submit such a request. We appreciate your receptivity to such a request.

Expectations on Remedy

We have found that it is always helpful in addressing contaminated waste sites to develop early tentative views of that which the eventual remedial program is likely to consist. That provides better focus, effectiveness and efficiency in organizing and conducting investigative and other related work. In this case, we believe that it is possible on the basis of the extensive data and analysis accumulated to date to project with reasonable confidence the principal features on an appropriate control program for these areas, as follows:

The main feature of the control program will be the installation of extraction wells at selected points along the riverfront to intercept any unacceptable migration of contaminants into surface waters, with appropriate related actions. Those would of course include arrangements for treatment of the extracted groundwater. We believe that the area between the various source areas and the riverfront should be used as an attenuation zone. Our preliminary analysis indicates that it would be impossible to remediate the groundwater throughout the plumes to the point that it would meet quality standards for public water supply at any point within the foreseeable future. Moreover, use of this area purely as an attenuation zone would not materially extend the eventual projection as to when such remediation could be accomplished.

We believe that further control measures at the source areas themselves, above and beyond those already completed, would be very limited since most such measures would be highly costly and would provide limited practical benefits. An appropriate set of engineering and institutional controls would be required in order to assure a prevention of access to any areas where risks might be encountered and to assure that contaminated groundwater would not be withdrawn for water supply purposes. In this regard it is helpful that the local communities have already adopted ordinances prohibiting any use of these areas for water supply. We also recognize that it will be necessary to obtain concurrence of the State of Illinois to this approach, specifically including use of the affected area for attentuation purposes.

Coordination with Other Authorities and Parties

We emphasized during the meeting our intent to work closely with representatives of the State of Illinois, the local governments of the villages of Cahokia and Sauget, and the general public to assure effective communication to all affected parties. We described our past and continuing efforts in this area, including our active support for local efforts to develop a theme park by the PARX Foundation that would give impetus to economic renewal in these communities. We are definitely committed to continuing these efforts and will keep you fully informed as to our efforts in that regard. We will also continue our diligent efforts to encourage participation by other PRP entities and to develop constructive joint efforts with such parties.

NPL Proposal for Areas 1 and 2

At the meeting we presented an overview of efforts we and others are encouraging to enhance economic opportunities in the region. Because of these efforts we are concerned with proposals for listing Area 1 and Area 2 as Superfund sites. Our concern centers on our belief that it would be a mistake to finalize the proposed NPL listings, since they are not needed to drive the work and additionally would adversely affect prospects for economic stabilization of the region and economic renewal in the region.

Conclusion

Once again we want to thank you for the constructive and helpful meeting on October 3. We hope this status summary will be helpful from your viewpoint. We look forward to continuing to work with you on these matters and would be pleased to respond to any questions or comments you may have.

Sincerely,

Brent J. Gilhousen

Brent J. Gilhousen
Assistant General Counsel
Environmental

cc: J. Quarles, Esq.

SOLUTIA - 062

Kenneth
Bardo/R5/USEPA/US
12/03/2001 11:58 AM

To rjhill1@solutia.com
cc
bcc murawski.richard@epa.gov

Subject Plume Capture Design

Bob - There are a couple of issues I wanted to follow up on.

I e-mailed you comments on the testing results and remedial action plan for MCB spill on July 31, 2001. I still have not received any specific responses to the comments. It is been nearly a year since the spill and it is uncertain how much of MCB free-product has been recovered. Also the feasible remedial alternatives to address the requirements of the RCRA 3008(h) Consent Order have not been defined by Solutia.

Solutia had also committed to submit to U.S. EPA a design to capture the identified groundwater contaminant plume before it discharges to the Mississippi River in an October 3, 2001 meeting and follow-up November 5, 2001 conference call. The design was to be submitted to U.S. EPA by the end of November. The design would be followed up by the submission of a focused feasability study by the end of the year as determined in U.S. EPA's November 14, 2001 Notification of Additional Work.

U.S. EPA has not yet received the design to capture the groundwater contaminant plume. I was also notified last week by the Superfund Division that the focused feasability study would not be submitted as required by the end of the year but would be submitted later in January 2002.

As you know, the RCRA 3008(h) Consent Order requires Solutia to stabilize the migration of contaminated groundwater at or from the facility by 1/1/2002. The design and focused feasability study are integral components of the process to address the stabilization of contaminated groundwater and need to be submitted to U.S. EPA for evaluation and comment. As provided for in Setcion IX.2 of the RCRA 3008(h) Consent Order, Solutia is subject to stipulated penalties of $5,000 per day for failure to adequately demonstrate that groundwater migration is stabilized by 1/1/2002.

Please let me know the status of the MCB spill and the submittals to address the groundwater contaminant plume. Thanks. - Ken

SOLUTIA - 064

UNITED STATES ENVIRONMENTAL PROTECTION AGENCY
REGION 5
77 WEST JACKSON BOULEVARD
CHICAGO, IL 60604-3590

DEC 17 2001

REPLY TO THE ATTENTION OF
SR-6J

Mr. Brent J. Gilhousen
Assistant General Counsel
Environmental
Solutia, Inc.
P.O. Box 66760
St. Louis, MO 63166-6760

Dear Mr. Gilhousen:

Thank you for your letter of October 31, 2001, which provided a status summary with regards to issues discussed at our meeting on October 3, 2001. At this meeting we discussed matters regarding the Administrative Order on Consent (AOC) entered into between the United States Environmental Protection Agency (U.S. EPA) and Solutia on May 3, 2000, under the Resource Conservation and Recovery Act (RCRA) program with respect to the Krummrich facility as well as the status of activities with respect to the Superfund proceedings concerning Sauget Area 1 and Sauget Area 2. U.S. EPA would like to take this opportunity to clarify its position on several issues discussed in your letter.

Your letter offers three specific suggestions for U.S. EPA to consider. The first suggestion recommends an integrated approach be taken regarding the control of contaminated groundwater in the vicinity of Site R. Recent field investigations suggest that groundwater contamination from the Krummrich facility, Sauget Area 1 and Sauget Area 2 commingle in the vicinity of Site R and ultimately discharge to the Mississippi River. U.S. EPA believes coordinating both RCRA and Superfund efforts to control the groundwater plumes in the vicinity of Site R is appropriate, and that an interim response action performed at Sauget Area 2 is the appropriate mechanism.

On November 14, 2001, U.S. EPA sent Solutia a letter requiring the submission of a focused Feasibility Study (FS) for a groundwater containment system to be installed in the vicinity of Site R. On December 3, 2001, U.S. EPA received a letter from Solutia which contained a preliminary design document of the extraction well system. This letter also stated that this design work would be part of the focused FS which is to be submitted in January 2002. Thereafter, it is U.S. EPA's intent that Solutia perform the implementation of the interim groundwater response action pursuant to an interim action Record of Decision (ROD). Compliance with the interim action ROD would satisfy Solutia's obligation pursuant to the RCRA AOC to demonstrate compliance with the Environmental Indicator for control of migration of contaminated groundwater. In addition, the parties would ensure that the interim action would be consistent with any subsequent final groundwater response action selected by U.S. EPA for Sauget Area 2.

In the December 3, 2001, letter, Solutia also requests a 90 day extension from the January 1, 2002, deadline to demonstrate compliance with the Environmental Indicator for control of migration of contaminated groundwater as required under the RCRA AOC. The time extension will be evaluated based on the effectiveness of the proposed design to control the discharge of contaminated groundwater to the Mississippi River as outlined in the preliminary design document received on December 3, 2001, and the adequacy of the focused Feasability Study required in U.S. EPA's November 14, 2001, letter. The time extension will also consider Solutia's continued cooperation in implementing the selected interim groundwater remedy, as well as its continued compliance with work required pursuant to the RCRA and Comprehensive Environmental Response, Compensation, and Liability Act (CERCLA) AOCs.

Your letter also proposes that U.S. EPA not proceed to make a final remedy selection or develop a Record of Decision (ROD) for Area 1 until the Remedial Investigation/Feasibility Study (RI/FS) for Area 2 and the RCRA Facility Investigation are complete which is anticipated to occur in 2004. U.S. EPA agrees that a final groundwater remedy for Area 1 should not be made until the sampling investigation for Area 2 and the Krummrich facility are complete. However, our intent to proceed in this manner is expressly contingent upon the expeditious implementation and success of the groundwater interim response action at Area 2 in preventing an unacceptable groundwater discharge to the Mississippi River.

In addition, it is U.S. EPA's position that control measures at the Sauget Area 1 source areas are necessary and appropriate at this time and the selected remedy must contain both engineered and institutional control components addressing these source areas. U.S. EPA plans on finalizing the Sauget Area 1 Engineering Evaluation/Cost Analysis (EE/CA)-RI/FS Report in the near future pending receipt and resolution of any Illinois EPA comments. An interim action ROD for the Sauget Area 1 source areas will likely be issued sometime in 2002.

The third suggestion provides for a single consolidated report to be submitted to satisfy the requirements for an RI/FS at Area 2 and for a corrective measures study for the Krummrich facility. Upon further evaluation, U.S. EPA does not encourage pursuing this approach. During preliminary negotiations between U.S. EPA and Solutia, U.S. EPA encouraged a consolidated effort between the three sites under Superfund authority. Solutia strongly objected to this approach and insisted that the Krummrich facility be addressed using RCRA authorities. Work at Sauget Area 2 and the Krummrich facility is now being conducted under different authorities, enforcement mechanisms, scopes of work, schedules, and performance standards. From an administrative standpoint, U.S. EPA believes it would be very difficult and resource intensive to try to consolidate the reports at this time, and such an effort would provide no benefit to the U.S. EPA or the public. From a technical perspective, a coordinated effort makes sense for addressing the commingled plume to prevent discharges to the Mississippi River, but does not seem reasonable for addressing the site-specific response actions necessary at each of the three areas.

Your letter also discusses Solutia's expectations for a final remedy at Sauget Areas 1 and 2 and the Krummrich facility. U.S. EPA finds Solutia's expectation on the final remedy to be somewhat worrisome and inconsistent with the Agency's goals for groundwater and the control of contaminated source areas. U.S. EPA's goals are discussed in the Advanced Notice of Proposed Rulemaking (ANPR) for Corrective Action found in the May 1, 1996, Federal Register, Volume 61, pp. 19431-19464, which states: "EPA expects to return usable groundwaters to their maximum beneficial uses whenever practicable within a time frame that is reasonable given the particular circumstances of the site." (61 FR 19448). The ANPR for Corrective Action further states: "EPA also expects to control or eliminate surface and subsurface sources of groundwater contamination." (61 FR 19448). It is too early to suggest that remediation of the groundwater throughout the plumes is impossible based on the limited available information and analyses.

U.S. EPA understands that complete groundwater restoration might be technically impracticable (TI). Solutia may apply for a TI waiver considering the engineering feasibility and reliability of attaining the media cleanup standards. Also, the remediation may be technically possible but the scale of the operations required might be of such a magnitude and complexity that the alternative would be impracticable (TI Guidance-OSWER Directive 9234.2-25). However, TI decisions should generally be made only after interim or full-scale remediation systems are implemented to evaluate the effectiveness of restoring groundwater. U.S. EPA's Subsurface Protection and Remediation Division of the National Risk Management Research Laboratory in Ada, Oklahoma, will be providing assistance with regards to the practicability of restoring groundwater in this area.

U.S. EPA further disagrees with Solutia's assessment that further source control measures would be "very limited" because such measures would be highly costly and would provide limited practical benefits. This position would be inconsistent with U.S. EPA's goals for the control of contaminated source areas as discussed in the ANPR for Corrective Action. Such a gross generalization regarding source area control measures is especially inappropriate given the nature of waste present in the area and that two of the three areas being discussed have not completed their investigations. Furthermore, TI guidance (OSWER Directive 9234.2-25) provides that source control measures be initiated even if Solutia were to present a demonstration that groundwater restoration is technically impracticable. In addition, certain source control measures will be necessary to ensure protection of human health and the environment and to control current human exposure to contamination (RCRA Environmental Indicator CA725). Therefore, source control measures should be considered necessary and should not be minimized as a future requirement by Solutia.

U.S. EPA is committed to working closely with Solutia and the other PRPs in developing remedies for Sauget Area 1, Sauget Area 2 and the Krummrich facility which are both practical and protective of human health and the environment. U.S. EPA reiterates its commitment to enhancing coordination between the RCRA and Superfund programs and will try to maintain consistency between the three areas especially with respect to groundwater containment and restoration remedies.

Sincerely,

Wm. E. Muno

William E. Muno, Director
Superfund Division

for Robert Springer, Director
Waste, Pesticides and Toxics Divison

cc: Sandy Bron, IEPA
Alan Faust, Solutia

SOLUTIA - 066

Certified Mail
#7099 3400 0000 9585 4243

December 27, 2001

DE-9J

Mr. Robert Hiller
Solutia Inc.
500 Monsanto Avenue
Sauget, IL 62206-1198

RE: Time Extension for Stabilization of
Groundwater at W.G. Krummrich Plant
Solutia Inc.
ILD 000 802 702

Dear Mr. Hiller:

The United States Environmental Protection Agency (U.S. EPA) has reviewed Solutia's request dated November 30, 2001, for a 90 day time extension to stabilize groundwater at the W.G. Krummrich Plant. The Administrative Order on Consent (AOC), Docket No. R8H-5-00-003 requires the migration of contaminated groundwater to be stabilized by January 1, 2002.

As part of the time extension request, Solutia submitted a *Discharge Control Study* and *Technical Specifications* for a groundwater extraction system to demonstrate its progress in designing a remedial system capable of capturing contaminated groundwater before it discharges to the Mississippi River, as required by the AOC.

Solutia proposes the construction of a three-well, hydraulic-barrier groundwater extraction system to control the discharge of contaminated groundwater to the Mississippi River. The collected groundwater would be conveyed back to the industrial sewer system at the W.G. Krummrich Plant. The extraction wells would be placed between the Mississippi River and the west edge of the Site R landfill to collect groundwater contaminated from suspected source areas located to the east, including the CERCLA Sauget Area 2 Sites. Therefore, the proposed interim remedial action to stabilize the migration of contaminated groundwater from the W.G. Krummrich Plant is also subject to approval and comment under CERCLA authority.

Pursuant to CERCLA authority, a focused feasability study (FS) was determined to be necessary by the U.S. EPA, Superfund Division on November 14, 2001, to address the migration of contaminated groundwater. Solutia submitted the FS on December 21, 2001. The work required under CERCLA authority requires additional time to complete, including a Record of Decision and public comment on the proposed hydraulic-barrier groundwater extraction system.

To address the CERCLA requirements described above, U.S. EPA, Waste, Pesticides, and Toxics Division approves a three-month time extension, to April 1, 2002, to stabilize groundwater as allowed for in Section VI of the AOC. Concurrent with the time extension, Solutia must adequately address the enclosed comments on the *Discharge Control Study* and *Technical Specifications* for the proposed hydraulic-barrier groundwater extraction system within 30 days of receipt of this letter. The enclosed comments address deficiencies in the information provided in the study which limits a proper evaluation of the accuracy of the groundwater model. In addition, constructive comments are provided on the specifications for the groundwater extraction system.

U.S. EPA is available to meet in January 2002 to discuss the comments to assure that the study and specifications result in an effective groundwater extraction system capable of preventing the discharge of contaminated groundwater to the Mississippi River that poses a threat to human health and the environment.

If you have any questions or would like to schedule a meeting, I can be reached at (312) 886-7566 or at bardo.kenneth@epa.gov.

Sincerely yours,

Kenneth S. Bardo

Kenneth S. Bardo, EPA Project Manager
Corrective Action Section

Enclosure

cc: Alan Faust (via E-mail), Solutia
Jim Moore, Illinois EPA
Gina Search, Illinois EPA

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Solutia Inc.
500 Monsanto Ave.
Sauget, IL 62206-1198

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ENCLOSURE

Discharge Control Study

General Comments

1. The report does not provide all the details necessary to evaluate the accuracy of the groundwater model. The assumptions, hydraulic properties, and results of the numerical model are not clearly discussed in the text. Specific subject areas for which the report does not provide adequate detail are listed below.

   - No definition of "significant" flow from the Mississippi River is provided with respect to the flow rate of the three pumping wells.

   - Calibration techniques are not fully discussed.

   - Sensitivity analyses are not discussed.

   - Storage values are not provided for the various modeled layers.

   - No discussion of model limitations is included.

2. The report does not clearly state whether the numerical groundwater flow model results are based on steady-state conditions, transient conditions, or both. Both steady-state and transient models should be used to accurately detail hydraulic conditions in the modeled area, or the report should discuss why one or the other type of model has not been developed.

3. The report does not clearly discuss model calibration and does not mention whether a sensitivity analysis was performed on the model. Commonly, sensitivity analyses are performed by changing hydraulic conductivity (K) values, storage parameters, recharge values, and boundary conditions and then determining the magnitude of changes in head throughout the modeled domain (Anderson and Woessner 1992).

4. The report does not mention the presence or absence of actual confining layers between the three modeled horizons, nor does the report give a leakance value for any confining layers that may be present.

Specific Comments

1. <u>Project Background, Page 2, Paragraph 1</u>. This paragraph discusses the site geology and hydrogeology and provides the elevations of the three modeled layers. The report should discuss the hydrogeologic properties of any confining layers that may be present in the saturated zone.

2. <u>Key MODFLOW Model Attributes, Assumptions, and Input Parameters, Page 4, Bullets 1 and 2</u>. These two bullets discuss the average river level stage and riverbed conductance values used to represent the Mississippi River in the groundwater model. The bullets do not mention whether the river was simulated using MODFLOW's river package feature or another method. The "Modeling Approach" section of the report only refers to the river simulation in terms of "river cells" and provides no further explanation as to how the river was represented. Clarify the method used to represent the river in the modeled domain.

3. <u>Key MODFLOW Model Attributes, Assumptions, and Input Parameters, Page 4, Bullet 5</u>. This bullet briefly discusses the regional pumping center along the north edge of the modeled domain. The pumping center is said to have a discharge rate of 4,167 gallons per minute (gpm). The bullet does not discuss which model layer or layers are being pumped by the pumping center. The bullet also does not mention whether the discharge rate of 4,167 gpm is constant or an average or whether there is any knowledge of the pumping continuing at this rate in the future. The report should discuss: 1) the effect of the pumping center on groundwater flow at the site; and 2) the expected activity at the pumping center in the future and its probable effect on groundwater flow at the site.

4. <u>MODFLOW Calibration, Page 4, Paragraph 1</u>. This section states that flow calibration was performed by adjusting the river level to 398.5 feet above mean sea level (amsl), which was the average river level during the 24-hour period preceding the midpoint of the sampling period. This value differs greatly from the average river stage value of 391 feet amsl stated in the "Key MODFLOW Model Attributes, Assumptions, and Input Parameters" section. No justification is provided in the report for selecting the value of 398.5 feet amsl for the calibration simulation. The report should explain the use of the two different river stage values.

5. <u>MODFLOW Calibration, Pages 4 and 5</u>. This section discusses the model calibration methods and results. Although the text cites Table 1, which compiles the results of the statistical analysis of the modeled and observed water level data, minimal discussion of these results is presented in the text. For example, the results of the statistical analysis included a root mean square (RMS) value of 3.19 for model layer 1. This value suggests a poor match of the modeled water level data to the observed water level data. The report states that "because of the small contribution to flow to the river from Layer 1, this match was considered to be acceptable." However, the large RMS value calculated for layer 1 may have resulted in significant modeling error. The report should expand the discussion of why this RMS value was acceptable. For example, the report could point out that an unconfined aquifer such as layer 1 is more difficult to accurately represent than a confined layer.

   In addition, this section discusses the K array used for each layer in the model. U.S. EPA assumes that a uniform K array was used for layer 1 because of a lack of spatial data; however, the K values for layers 2 and 3 varied laterally across the modeled domain. According to Table 1 of the report, layers 2 and 3 actually had fewer data points on which the spatial variation of K could be based. The report should discuss why K values varied spatially in layers 2 and 3 but were uniform in layer 1.

   This section also discusses the calibration of modeled layer 1. To better match simulated hydraulic head values to observed values, the uniform K array of layer 1 was reduced from 0.01 to 0.0005 centimeters per second, a change of nearly two orders of magnitude. The report does not discuss performance of a sensitivity analysis to determine the effect of a large reduction in K. The reduction in K may have resulted in layer 1 appearing to contribute little flow to the river. The report should discuss the impact of reducing K in layer 1.

6. <u>Modeling Approach, Page 5, Paragraph 2</u>. This paragraph discusses determination of the flow rate of contaminated groundwater to river cells in layers 1 and 2. The paragraph does not fully discuss the hydraulic and physical attributes of the river cells used in the model. To fully conceptualize the hydraulic and physical properties of the river cells, information such as the length of each river reach, the width of the river, and the thickness of the riverbed should be provided (McDonald and Harbaugh 1988). The report should include this information.

7. **Modeling Approach, Page 5, Paragraph 2.** This paragraph discusses evaluation of different flow control pumping schemes. The report states that the "most vulnerable location for river flow inflow to a Site R flow control well was determined." The report does not explain what "vulnerable" means in the context of flow rates from the river and discharge rates from the pumping well. Also, the report does not discuss how the "most vulnerable location" was determined. The report should clarify these matters. In addition, the paragraph refers to a "critical well" at which the discharge rate was increased to determine the pumping rate that would cause inflow from the river. The report does not clearly state which well is the "critical well," where this well is screened, or where this well is located. The report should clarify these matters as well.

8. **Modeling Results, Page 5, Paragraph 2.** This paragraph states that the maximum sustainable pumping rate that does not result in inflow from the river is between 200 and 250 gpm. U.S. EPA assumes that this range is based on the discussion in the "Modeling Approach" section; however, the report does not clearly explain how the range of 200 to 250 gpm was determined. The report should explain the determination of these values.

9. **Figure 1.** This figure does not contain a legend that defines the various colored zones in the modeled area. The figure should include a more complete legend that defines these zones. Also, this figure does not identify the locations of any confining layers in the saturated zone. The locations of any confining layers present should be identified in the figure.

10. **Figure 5.** This figure depicts the locations of river discharge zones and discharge control wells for the three modeled horizons in the study area. Layers 2 and 3 appear to be mislabeled in the figure.

11. **Attachment C.** This attachment contains a U.S. Army Corps of Engineers (U.S. ACE) map of the Mississippi River (river miles 178.2 to 180.3) and is represented as being in the vicinity of Site R. However, a more appropriate U.S. ACE river map containing Site R— that for river miles 176.2 to 178.2— is not included. The attachment should include the U.S. ACE river map containing Site R (river miles 176.2 to 178.2) and the appropriate hydrographic data from this map used as input in the MODFLOW model.

REFERENCES

Anderson, M.P., and W.W. Woessner. 1992. *Applied Groundwater Modeling*. Academic Press. San Diego, California.

McDonald, M.G., and A.W. Harbaugh. 1988. "A Modular Three Dimensional Finite-Difference Ground-Water Flow Model." *Techniques of Water Resources Investigations of the United States Geological Survey*. Book. 6. Chapter. A1.

Technical Specifications

Specific Comments

1. Page 7, Section 2.2.1, Well Casing Pipe. The text states that well casing pipe must be "10-inch I.D. low carbon stainless steel." However, the type of stainless steel and the thickness of the casing are not specified. Typically, specifications include this type of information. Low-carbon stainless steel is usually Type 316L. Also, the diameter of stainless steel pipe is usually identified in terms of outside diameter. The text should include this missing information.

2. Page 7, Section 2.2.2, Grout, Part A. The text states that "neat cement grout" consists of "cement and water in proportion of 1 bag (94 lb) Portland cement to 8.3 gal clean water." Part B, however, states that the mix design "shall be approved by the REMEDIAL DESIGNER." It is not clear why the remedial designer has to approve the mix design when it is presented in Part A. Also, the type of cement required is not clear. The text should identify the type of cement required for the grout and clarify the mix design requirements.

3. Page 8, Section 2.2.3, Screen, Part B. The text specifies that the screen must be Type 304 stainless steel with a nominal diameter of 10 inches. Because low-carbon stainless steel is specified for the well casing pipe, it is advisable to use the same type of stainless steel for the well screen. Type 304 stainless steel has a carbon content of 0.08 percent, which is almost three times greater than the carbon content of the stainless steel specified for the well casing pipe. The material requirements for the screen should be reviewed in light of this information.

4. Page 9, Section 2.2.6, Part C. The text calls for a "steel pitless case of the same size as [the] well casing, with black corrosion resistant coating." It is not clear why a corrosion-resistant coating is required; if stainless steel is used, corrosion should not be a significant problem. Also, it is not clear what type of "black corrosion resistant coating" is required. In addition, the material of construction for the pitless adapter is not specified. The text should clarify these matters.

5. Page 9, Section 2.2.7, Galvanized Steel Drop Tubing. It is not clear why galvanized steel tubing is specified for what appears to be a well pump discharge. Also, the thickness of the tubing is not specified. If the installation is designed for 30 years of useful life, stainless-steel, schedule 40 pipe should be specified as the discharge pipe material. Typically the well pump is supported

by the discharge piping, as it is the pump's and piping's weight that maintains the seal in the pitless adapter. The text should be revised in light of this information and the construction material and the class or schedule of the discharge piping should be specified.

6. <u>**Page 10, Section 2.3.3, Part B**</u>**.** Phrasing such as "it is suggested that" should be avoided. The specifications should be clear and concise regarding the drilling method to be used for installation of wells. If necessary, the specifications should include a provision for the contractor to propose an alternative drilling method that can be accepted or rejected by the engineer.

7. <u>**Page 10, Section 2.3.4, Part B**</u>**.** The text implies that boreholes will be sampled during drilling; however, the sampling intervals and method are not specified. The text should clearly state the sampling intervals and procedures required.

8. <u>**Page 11, Section 2.3.5, Part H**</u>**.** The text requires the contractor to conduct a short- duration performance test for each well. It is not clear what the objective of this test is or what will determine acceptance or rejection of a well. The text should be clearly state the test objective and the criteria for well acceptance.

9. <u>**Page 11, Section 2.3.6, Part A**</u>**.** This section calls for decontamination of drilling equipment when it arrives on site and before it leaves the site. It is not clear, however, whether down-hole drilling equipment will be decontaminated between boreholes. The text should clarify this matter.

10. <u>**Page 12, Section 2.3.7, Part A**</u>**.** The text discusses collection and containerization of liquids generated during well installation. However, it is not clear whether these liquids will be sampled for analysis or how they will be disposed of. The text should clarify this matter.

11. <u>**Page 14, Section 3.1.3, Part B, Subpart 1**</u>**.** The specification requires submittal of a "pump manufacturer's statement of overall efficiency guarantee for [the] pumping unit under specified conditions." The conditions are not clearly specified. The text should specify the conditions if such a guarantee is to be required.

12. <u>**Page 14, Section 3.1.6, Parts A, B, and C**</u>**.** If it is expected that pumping conditions will vary greatly, it would be prudent to specify pumps with variable-frequency drives. Such pumps would accommodate a range of flow

rates and any future adjustments required by fluctuations in groundwater levels caused by pumping or seasonal factors. Dropping groundwater levels will increase the static head, reducing the pumping rate required. Pumps with variable-frequency drives can provide the desired discharge rate regardless of changes in static head. The text should be reviewed in light of this information and revised as necessary.

13. Page 15, Section 3.2.2, Part D. The specification states that "pumps shall be sized to provide at least 220 gallons per minute (plus or minus 20%) flow rate against a total head of at least 70 feet, depending on final design parameters for the conveyance system." This is an unusual requirement. Typically, pump specifications state the required pumping rate at a fixed total dynamic head for constant-speed pumps. The plus or minus 20 percent allowance for the flow rate might allow the contractor to choose between two pump sizes, and the contractor would probably furnish the smaller or less expensive pump, which might be too small in the long run. The text should be reviewed based on these considerations and revised as necessary.

14. Page 15, Section 3.2.3. According to Drawing No. 3, it appears that the check valve will be located in each well on top of the well pump. This placement of the check valve may cause maintenance problems, as the valve would not be accessible. Also, high shutoff head may cause the valve to fail prematurely as a result of flow reversal. Because the drawings indicate use of valve vaults, it may be advisable to install the check valves in these vaults, where they will be easily accessible for maintenance and will not be exposed to high shutoff head. Additionally, check valves for such installations are usually specified to be of stainless-steel construction. The design should be reviewed in light of these considerations, and the specification should be revised as necessary.

15. Page 16, Section 3.2.4, Part B. The text requires the contractor to "provide flow control valves to prevent flow rates above [the] operating range of [the] well pump." However, it is not clear what this operating range is. It is also not clear what types of valves are required or what their materials of construction are to be. Use of pumps equipped with variable-frequency drives would eliminate the need for these valves (see comment no. 12). The design should be reviewed in light of these considerations, and the specification should be revised as necessary.

16. <u>Page 17, Section 3.2.7, Part E</u>. The text should be revised to read as follows: "Pump motors shall be non-overloading throughout their entire operating range."

17. <u>Page 27, Section 5.2.2, Part B</u>. The text states that the pumps will be operated by level switches located in each extraction well. However, the operating range of the switches and the distance between the switches in each well are not specified. Also, it is not clear whether fluctuations in groundwater levels will be addressed by the control scheme. These matters should be clarified. In addition, the text indicates that a high high-level switch will initiate a remote alarm in the Department 277 control room, which will be more than 6,000 feet away from one extraction well. The remote alarm is not shown on the drawings, and therefore it is not clear whether this alarm will be hard-wired or activated via an autodialer. The text and drawings should be revised to clarify this matter.

18. <u>Drawing No. 2</u>. This drawing shows what appears to be a single force main to which the three well pump discharges will be connected. It is difficult to evaluate this system because no pipe sizes are included on the drawings or in the specifications. This information should be shown on the drawings to facilitate the review process and avoid confusion during bidding. Drawing No. 2 also shows electrical lines to be underground electrical feeders. However, the conductor sizes required are not shown. This information should be provided on the drawing. It should be noted that running a long feeder from the pole barn to the well located furthest south will likely produce a voltage drop because of the distance involved (about 2,000 feet).

19. <u>Drawing No. 3</u>. The specification in Section 3.2.6 calls for a submersible cable with at least 5 extra feet available for termination in a junction box at the "pump head, or wellhead." Drawing No. 3 does not indicate that the submersible cable will terminate in a junction box at the wellhead. Rather, the drawing indicates that the submersible cable will enter the electrical conduit through the well casing and run underground to a junction box just below the control panel; this is the only junction box shown. The discrepancy between the text and drawing should be reconciled. The drawing also indicates that power cables and flow sensor telemetry will be installed in the same schedule 40 polyvinyl chloride conduit. Installing power and telemetry wiring in one conduit is not recommended. This design element should be reviewed and revised as necessary.

20. <u>**Drawing No. 4.**</u> It is not clear why manhole steps are required in the vault shown. The vault will be only 3.5 feet deep and will have a sampling port located under the 30-inch-square access cover, making it impossible to enter. Also, the 3-inch ball valve downstream from the sampling line will be difficult to operate as it is presently configured. In addition, flow sensor wiring should be terminated in a waterproof junction box. Moreover, a provision such as a French drain should be included in the design to remove accumulated rainwater. The drawing should be reviewed in light of these considerations and revised as necessary.

SOLUTIA - 068

SOLUTIA

Applied Chemistry, Creative Solutions

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January 28, 2002

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Re: Response to Comments on the Design of Wells
Focused Feasibility Study, Sauget Area 2 Sites

Dear Mr. Bardo and Mr. Ribordy

Enclosed are Solutia's response to comments received from your offices on wells designed to be installed at Site R in Sauget. These wells are proposed for use in the Focused Feasibility Study for the Sauget Area 2 Sites, which was submitted to U.S. EPA on December 21, 2001. If you have any questions regarding the wells, please give me or Bruce Yare a call.

Sincerely Yours

Alan Faust

Alan Faust

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Solutia-068

Groundwater Discharge Control System

W.G. Krummrich, Sauget, Illinois

Design Basis and Design

Response to Comments

DRAFT

January 28, 2002

Submitted To:

USEPA Region 5, Chicago, Illinois

Submitted By:

Solutia Inc, St. Louis, Missouri

Response to Comments - Design Basis

General Comments

Specific Comments

Response to Comments - Design

Specific Comments

Attachments

General Comments

Comment No. 1

The report does not provide all the details necessary to evaluate the accuracy of the groundwater model. The assumptions, hydraulic properties, and results of the numerical model are not clearly discussed in the text. Specific subject areas for which the report does not provide adequate detail are listed below.

- **No definition of "significant" flow from the Mississippi River is provided with respect to the flow rate of the three pumping wells.**

Response:

"Significant flow" is the flow rate that would induce recharge from the Mississippi River for a groundwater migration control well placed where the Mississippi River is the closest to the downgradient boundary of Sauget Area 2 Site R.

As shown in Figure 6 of the Discharge Control Study (Attachment 1), a well at this location has a limited impact on the equipotential line that is aligned with the eastern boundary of the river. A qualitative review of this pumping scheme indicated that most of the flow into the well at this pumping rate was from upgradient portions of the water-bearing unit, and not from the river. Model runs with higher pumping rates showed that the equipotential line on the river was captured by the well resulting in significant flow from the river to the well.

Definition of "significant flow" is important because the groundwater migration control system is designed to maximize recovery of impacted groundwater discharging to surface water and minimize the infiltration of recharge from the river.

- **Calibration techniques are not fully discussed.**

Response:

Model Calibration - The Mississippi River stage value of 398.5 ft amsl was a representative value of the river stage during the water level monitoring event conducted on October 25, 2001. This was the event that was used to calibrate the model. A river stage of 3985 ft amsl is higher than average river stage.

The initial hydraulic conductivity values used for the calibration process were 0.01 cm/sec for the Shallow Horizon (Layer 1 in the model) and a variable hydraulic conductivity field obtained from Schicht (Attachment 2) for the Middle and Deep Horizons (Layers 2 and 3 in the model).

Shallow Layer Calibration - The shallow layer value was taken from modeling studies performed for the Sauget Area 1 EE/CA and RI/FS (Attachment 3). This value was a conservative (high-end) hydraulic conductivity estimate that was partially based on slug tests conducted at Sauget Area 1 Site I that showed a hydraulic conductivity value of $4.5x10^{-3}$ cm/sec.

Initial calibration runs showed that the predicted static water level from the shallow layers were considerably lower than the actual values measured on Oct. 25, 2001. By decreasing both the horizontal and vertical hydraulic conductivity arrays in the model, a better match was achieved. The final calibration result for the shallow layer was a constant value of $5x10^{4}$ cm/sec.

Additional data available for the Discharge Control Study (Attachment 1) suggested that this lower hydraulic conductivity of Layer 1 was appropriate. First, geologic cross-sections developed for Site R by URS in 2001 (Attachment 4) indicated that the shallow layer was comprised primarily of clay. Second, Geraghty and Miller reported that slug test values for the Shallow Hydrogeologic Unit at Sauget Area 2 Site R ranged from $9x10^{-5}$ cm/sec to $6x10^{-3}$ cm/sec in two studies completed in 1993. Geraghty & Miller also indicated that this unit was a "low permeability zone with fine-grained silty sand deposits predominating." These two studies are included as Attachment 5 and Attachment 6. Third, a review of the large-scale geologic cross-sections of the American Bottoms prepared by Bergstrom and Walker (Attachment 7), shows the upper portion of the cross section being largely comprised of fine-grained material.

Middle and Deep Horizon Calibration - The hydraulic conductivity map developed by Schicht (Attachment 2) was used for initial values of the horizontal hydraulic conductivity for the middle and deep layers (K_x and K_y, no anisotropy was assumed in the horizontal plane). Zones between lines of constant hydraulic conductivity were assumed to be arithmetic averages of the two hydraulic conductivities shown on the contour lines. For example, the initial hydraulic conductivity of the zone between the 3000 gpd/ft^2 and the 2500 gpd/ft^2 was assumed to be 2750 gpd/ft^2, or 0.13 cm/sec. The zone inside the 3000 gpd/ft^2 closed contour was assumed to have a hydraulic conductivity of 3250 gpd/ft^2, or 0.15 cm/sec. The initial estimate of vertical hydraulic conductivity (K_z) was 20% of K_x and K_y.

The initial calibration runs indicated that the hydraulic gradient between the portions of the middle and deep layers near and under the river was greater in the model than was represented in the data. Therefore, changes were made in the following order:

1. The zone between the 2500 gpd/ft^2 and 3000 gpd/ft^2 on Schicht (1965) (Attachment 1) (now labeled "0.137 cm/sec" for K_x and K_y on Figure 3) was extended entirely across the River in the area west of Site R.

2. The K_x and K_y (horizontal hydraulic conductivity) of the same zone were increased from 0.13 cm/sec to 0.137 cm/sec.

The hydraulic gradient between the middle and deep layers was greater in the model than in the Oct. 25, 2001 dataset. Therefore, changes were made in the following order:

1. The vertical hydraulic conductivity of all zones in both the middle layer and deep layer (K_z) was increased from an initial value of 0.20 of Kx and Ky, to a value of 0.50 of K_x and K_y to reduce the modeled head loss.

2. The constant head elevations on the boundary cells on the east, north, and south sides of the model were adjusted to match "steady-state" data developed by Clark (1997).

- **Sensitivity analyses are not discussed.**

Response:

Sensitivity analyses were conducted for the following parameters: recharge (high and low), upper layer conductivity (high and low), river stage and overall conductivity. The range that was varied for each parameter was based on ranges in the underlying data for each parameter used in the sensitivity analysis. Table 1 below summarizes the discharge from Site R to the river when various parameters are altered.

SENSITIVITY RUN DESCRIPTION	PERCENTAGE OF BASELINE (650 gpm) (%)
BASELINE CASE (From original report)	100%*
HIGHER Hydraulic Conductivity In All Three Layers. (K_x, K_y, and K_z shown in Fig. 3 increased by factor of 1.5).	130%
LOWER Hydraulic Conductivity In All Three Layers. (K_x, K_y, and K_z shown in Fig. 3 reduced by factor of 1.5).	78%
HIGHER Hydraulic Conductivity In Shallow Layer Only. (K_x, K_y, and K_z shown in Fig. 3 increased by factor of 10).	100%
LOWER Hydraulic Conductivity In Shallow Layer Only (K_x, K_y, and K_z shown in Fig. 3 reduced by factor of 10).	100%
HIGHER RECHARGE. Recharge Increased from 7.8 inches/yr to 9.9 inches per year.	119%
LOWER RECHARGE. Recharge Decreased from 7.8 inches/yr to 6.3 inches per year.	87%
HIGHER River Stage. River Stage Increased from 391 ft msl to 400.8 ft msl (the high monthly average flow).	25%
LOWER River Stage – River Stage Decreased from 391 ft msl to 383.1 ft msl (the low monthly average flow).	135%
HIGHER River Conductance, River Conductance multiplied by 2.7.	100%
LOWER River Conductance. River Conductance divided by 1.4.	100%

* Note that the reported value of 650 gpm includes 115 gpm (i.e., ~20%) of excess flow to account for unknowns in the modeling process. In other words, the actual discharge to the river under the baseline condition predicted by the model is 535 gpm (this discharge estimate was inadvertently omitted from the original report). The value of 650 gpm has been retained to address flow variability issues and modeling unknowns such as the ones indicated by the sensitivity analysis.

Conclusions that can be drawn from this sensitivity analysis are:

- Estimated groundwater discharge to the Mississippi River is insensitive to the hydraulic conductivity of the Shallow Hydrogeologic Unit;
- Underestimation or overestimation of the hydraulic conductivity of all three hydrogeologic units by a factor of 1.5 could result in groundwater discharges to the Mississippi River ranging from 510 to 850 gpm, respectively;
- A decrease in recharge of 2.5 inches/year or an increase in recharge could result in groundwater discharges to the Mississippi River ranging from 570 to 770 gpm, respectively;
- An increase in river stage of 9.8 ft. or a decrease in river stage of 7.9 ft. could result in groundwater discharges to the Mississippi River ranging from 160 gpm to 880 gpm; and
- Estimated groundwater discharge to the Mississippi River is insensitive to the conductance of the river bottom.

Sensitivity analysis indicates that groundwater discharge to surface water could vary between 160 and 880 gpm. A pumping rate of 650 gpm was selected at the design basis for the groundwater migration control system because this is the amount of water that can be pumped without inducing recharge from the Mississippi River.

A three-well, hydraulic barrier groundwater system with a total pumping capacity of 200 to 650 gpm should control discharge of impacted groundwater to surface water. Ability of this system to achieve remedial objectives and performance standards will be assessed by conducting a long-term step test at pumping rates of 200, 350, 500 and 650 gpm. Each step test will be run long enough to determine whether or not performance standards can be achieved. Duration of each step test is likely to be three months or longer. The goal of this testing is to empirically determine the amount of water that needs to be pumped to achieve performance standards.

Modeling (Attachment 8) indicates pumping rates of 200, 350, 500 and 650 gpm will capture 35, 60, 76 and 85 percent, respectively, of the mass loading to the Mississippi River. The amount of mass removal needed to achieve remedial objectives and performance standards is not known. Remedial objectives included in the Sauget Area 2 Focused Feasibility Study submitted to USEPA on December 21, 2001 are listed below:

- Prevent or abate actual or potential exposure to nearby human populations (including workers), animals or the food chain from hazardous substances, pollutants or contaminants;
- Prevent or abate actual or potential contamination of drinking water supplies and ecosystems;
- Achieve acceptable sediment toxicity levels, or range of levels, for all applicable exposure routes; and
- Mitigate or abate other situations or factors that may pose threats to public health, welfare or the environment.

Installation of a hydraulic barrier will achieve these objectives by reducing the mobility of groundwater contaminants by providing hydraulic control and removal of affected groundwater before it discharges to the Mississippi River downgradient of Sauget Area 2 Sites O, Q (Dog Leg), R and S; Sauget Area 1 Sites G, H, I and L; the W.K. Krummrich plant and other industrial facilities in the Sauget area. In the long term, the toxicity and volume of groundwater contaminants will also be reduced through the action of natural processes, such as biodegradation, adsorption, dilution, volatilization and chemical reactions with subsurface materials, occurring between the source areas and the hydraulic barrier and by removing and treating impacted groundwater migrating to the Mississippi River.

Sediment toxicity testing was selected in the Sauget Area 2 Focused Feasibility Study as an appropriate performance standard for assessing effectiveness of the hydraulic barrier because it is a direct measure of the impact of groundwater discharging to surface water downgradient of Sauget Area 2 Sites O, Q (Dog Leg), R and S; Sauget Area 1 Sites G, H, I and L; the W. G. Krummrich plant and other industrial facilities in the Sauget area. For that reason, sediment toxicity was considered the primary measure of effectiveness of the remedial action.

Impacts due to the discharge of groundwater to surface water are confined to an area approximately 2000 feet long (coinciding with the north and south boundaries of Sauget Area 2 Site R) and 300 feet from shore immediately downgradient of Site R. To monitor the effectiveness of the implemented remedy, sediment samples will be collected at three locations on four transects in this "impact zone". One transect will be located downgradient from each of the monitoring well clusters. Sediment samples will be collected 50, 150 and 300 feet from shore. Sediment toxicity tests (acute and chronic) will be performed on these samples to determine whether or not the implemented remedy is protecting the Mississippi River. Rather than performing separate acute and chronic tests, a single toxicity test will be performed on each sediment sample. If there are no observed acute effects, then the sediment toxicity test will be continued long enough to establish whether or not the samples exhibit chronic toxicity.

Groundwater-quality monitoring will include installation of a permanent groundwater-monitoring network and periodic monitoring to demonstrate compliance with objectives. The groundwater-monitoring program will be used to demonstrate groundwater plume stability (i.e. control discharge of impacted groundwater to the Mississippi River with concentrations high enough to cause sediment toxicity), future reductions in constituent concentrations in groundwater discharging to the Mississippi River (mass loading) and eventual compliance with remedial action objectives. The permanent groundwater-monitoring well network will consist of monitoring wells screened in the shallow, intermediate and deep groundwater zones at four separate locations between the hydraulic barrier and the Mississippi River, a total of twelve monitoring wells.

Figure 1 shows the proposed sediment and groundwater sampling locations.

- **Storage values are not provided for the various modeled layers.**

Response:

Steady-state runs were performed, and therefore no storage values were used in the model described in the report. Based on Geraghty and Miller (1993), representative storage coefficient values range from 0.04 to 0.10.

- **No discussion of model limitations is included.**

Response:

The model has the following key limitations:

- the shallow layer is assumed to have a constant hydraulic conductivity;
- the river is simulated with idealized cross section and river bottom conductance values; and
- only one river stage was evaluated after calibration.

As discussed above, estimated groundwater discharge to surface water is insensitive to the hydraulic conductivity of the shallow layer and river bottom conductance. Estimated groundwater discharge is sensitive to river stage. For high monthly average river flow and low monthly average river flow, estimated groundwater discharge is 160 and 880 gpm, respectively.

Comment No. 2

The report does not clearly state whether the numerical groundwater flow model results are based on steady-state conditions, transient conditions, or both. Both steady-state and transient models should be used to accurately detail hydraulic conditions in the modeled area, or the report should discuss why one or the other type of model has not been developed.

Response:

The groundwater flow model described in the report was based on steady-state conditions. A transient model was not performed because results from the 1993 Geraghty and Miller modeling study (Attachment 6) indicated that transient modeling resulted in only minor changes in their steady-state model results.

Comment No. 3

The report does not clearly discuss model calibration and does not mention whether a sensitivity analysis was performed on the model. Commonly, sensitivity analyses are performed by changing hydraulic conductivity (K) values, storage parameters, recharge values, and boundary conditions and then determining the magnitude of changes in head throughout the modeled domain (Anderson and Woessner 1992).

Response:

Model calibration and a subsequent sensitivity analysis are discussed in the response to General Comments 1 and 2.

Comment No. 4

The report does not mention the presence or absence of actual confining layers between the three modeled horizons, nor does the report give a leakance value for any confining layers that may be present.

Response:

The Shallow Hydrogeologic Unit was assumed to be unconfined with a porosity of 0.30. Hydraulic conductivity values indicate that the Shallow Hydrogeologic Unit serves as a semi-confining layer for the Middle and Deep Hydrogeologic Units. As shown in Figure 1 of Attachment 1, the potentiometric surface of the Middle Hydrogeologic Unit extends into the Shallow Hydrogeologic Unit, indicating confined or semi-confined conditions. No aquitards restrict vertical groundwater flow between the Middle and Deep Hydrogeologic Units.

Vertical hydraulic conductivity values were used in the model to calculate leakance terms. For the Shallow Horizon, $K_z = 1x10^{-6}$ cm/sec was used. For the Middle and Deep Horizons, $K_z = 0.50\ K_x$. These values were derived during the calibration process described above.

Specific Comments

Comment No. 1 **Project Background, Page 2, Paragraph 1**

This paragraph discusses the site geology and hydrogeology and provides the elevations of the three modeled layers. The report should discuss the hydrogeologic properties of any confining layers that may be present in the saturated zone.

Response:

Figure 1 of Attachment 1 shows the elevations of the three layers in the model. The shallow layer (Layer 1) extended from the water table to 395 ft msl. The middle layer (Layer 2) extended from 395 ft msl to 350 ft msl. The deep layer (Layer 3) extended from 350 ft msl to bedrock as determined by kriging the bedrock elevation map developed by Bergstrom and Walker (Attachment 7).

The vertical flow properties of the three layers are described in the response to General Comment 4. As noted in this response, the Shallow Hydrogeologic Unit is the only layer used in the model that acts as a confining layer. There is no aquitard between the Middle Hydrogeologic Unit and the Deep Hydrogeologic Unit. Therefore, the model does not include a confining unit between the middle layer and deep layer.

Comment No. 2 **Key MODFLOW Model Attributes, Assumptions, and Input Parameters Page 4, Bullets 1 and 2**

These two bullets discuss the average river level stage and riverbed conductance values used to represent the Mississippi River in the groundwater model. The bullets do not mention whether the river was simulated using MODFLOW's river package feature or another method. The "Modeling Approach" section of the report only refers to the river simulation in terms of "river cells" and provides no further explanation as to how the

river was represented. Clarify the method used to represent the river in the modeled domain.

Response:

Although not mentioned in the report, the river was modeled using MODFLOW's river package. Each river cell was assigned a river elevation, which was assumed constant for all river cells in the model, a bottom elevation, assumed to be the same for all river cells in the model and based on a single U.S. Corps of Engineers bathymetric cross section near Site R (Attachment 1), and a conductance term, which was derived from the average of monthly conductance estimates reported by Schicht (Attachment 2).

Comment No. 3 Key MODFLOW Model Attributes, Assumptions, and Input Parameters Page 4, Bullet 5

This bullet briefly discusses the regional pumping center along the north edge of the modeled domain. The pumping center is said to have a discharge rate of 4,167 gallons per minute (gpm). The bullet does not discuss which model layer or layers are being pumped by the pumping center. The bullet also does not mention whether the discharge rate of 4,167 gpm is constant or an average or whether there is any knowledge of the pumping continuing at this rate in the future. The report should discuss: 1) the effect of the pumping center on groundwater flow at the site; and 2) the expected activity at the pumping center in the future and its probable effect on groundwater flow at the site.

Response:

Regional pumping for highway dewatering in East St. Louis was assumed to be withdrawn from all three layers. Figure 4 of the Discharge Control Study (Attachment 1) indicates that the highway dewatering has little effect on the site . Head equipotential lines are relatively parallel to the Mississippi River near Site R, and do not curve north towards the pumping center until they get closer to the pumping center. Since this dewatering system is associated with Interstate 64, it is reasonable to assume that pumping will continue indefinitely. Therefore, future effects on groundwater flow at Site R due to this pumping center will continue to be the same as they are today, i.e. negligible.

Comment No. 4 **MODFLOW Calibration, Page 4, Paragraph 1**

This section states that flow calibration was performed by adjusting the river level to 398.5 feet above mean sea level (amsl), which was the average river level during the 24-hour period preceding the midpoint of the sampling period. This value differs greatly from the average river stage value of 391 feet amsl stated in the "Key MODFLOW Model Attributes, Assumptions, and Input Parameters" section. No justification is provided in the report for selecting the value of 398.5 feet amsl for the calibration simulation. The report should explain the use of the two different river stage values.

Response:

The Mississippi River stage value of 398.5 ft amsl was an average of hourly river stage values between 12:00 pm on Oct. 24 and 12:00 pm on Oct. 25. Preliminary model runs indicated that the response time for the near-river middle and deep layers to changes in Mississippi River elevation had timescales of hours (as opposed to days or weeks). Therefore an average river elevation for the 24 hours prior to the midpoint of the sampling event on Oct. 25 was selected. Oct. 25 was selected for calibration because: i) the data were representative of recent conditions, and ii) the data were readily available.

For the calculation of groundwater discharge from the site to the river, the model was run with the average river level stage of 391ft MSL. This value is the average river stage based on data between 1933 and 2001. The 50-year mean (1941-1990) river elevation is 390.30 ft MSL.

The November 1990 potentiometric surface map (Attachment 9) was developed from data taken when the Mississippi River stage was fairly low, around 385 ft msl. This value was selected as it covered the entire model area and was relatively recent. The 1990 map was compared to the calibrated water level map shown on Figure 4 of the Discharge Control Study (Attachment 1). In general, the shapes of the potentiometric surface for the middle and deeper layers were similar to the 1990 map. The predicted values (Attachment 1, Figure 4) did not provide an absolute match to the observed values (Attachment 9, Figure 14) due to differences in river stage between modeled and observed conditions.

Comment No. 5 **MODFLOW Calibration, Pages 4 and 5**

This section discusses the model calibration methods and results. Although the text cites Table 1, which compiles the results of the statistical analysis of the modeled and observed water level data, minimal discussion of these results is presented in the text. For example, the results of the statistical analysis included a root mean square (RMS) value of 3.19 for model Layer 1. This value suggests a poor match of the modeled water level data to the observed water level data. The report states that "because of the small contribution to flow to the river from Layer 1, this match was considered to be acceptable." However, the large RMS value calculated for Layer 1 may have resulted in significant modeling error. The report should expand the discussion of why this RMS value was acceptable. For example, the report could point out that an unconfined aquifer such as Layer 1 is more difficult to accurately represent than a confined layer.

Response:

Modeling focus on the middle and deep layers was warranted because:

- Sensitivity analysis indicates a change of less than 1 gpm when the hydraulic conductivity of the shallow layer is increased;
- Upper-range transmissivity of the shallow layer is 425 ft^2/d, (0.01 cm/sec x 15 ft thickness), 80 times less than the middle and deep layer's transmissivity of 35,000 ft^2/d (0.137 cm/sec x 90 ft) ;
- Actual flow contribution from the shallow lorizon may be less as the saturated thickness near the river is relatively small; and
- Modeling an unconfined, near-surface layer is more difficult than modeling a confined layer.

This focus is validated by comparison of predicted versus observed water levels in water level measurement wells located at Site R (Figure 2).

In addition, this section discusses the K array used for each layer in the model. U.S. EPA assumes that a uniform K array was used for layer 1 because of a lack of spatial data; however, the K values for Layers 2 and 3 varied laterally across the modeled domain. According to Table 1 of the report, Layers 2 and 3 actually had fewer data points on which the spatial variation of K could be based. The report should discuss why K values varied spatially in Layers 2 and 3 but were uniform in layer 1.

Response:

Data from Schicht (1965) were available to construct a detailed, spatially varying, hydraulic gradient array for the entire model area for the middle and deep layers. There were no maps of the shallow layer hydraulic conductivity over the entire scale of the model.

For the model, the shallow layer was assumed to have a constant hydraulic conductivity because: 1) no map was available that allowed preparation of a model-wide hydraulic conductivity array, 2) the shallow layer's apparent small contribution to flow due to a transmissivity 80 times lower than the middle and deep layers and 3) sensitivity analysis results that indicate an increase in shallow layer transmissivity results in a flow increase of less than one gpm

This section also discusses the calibration of modeled Layer 1. To better match simulated hydraulic head values to observed values, the uniform K array of Layer 1 was reduced from 0.01 to 0.0005 centimeters per second, a change of nearly two orders of magnitude. The report does not discuss performance of a sensitivity analysis to determine the effect of a large reduction in K. The reduction in K may have resulted in Layer 1 appearing to contribute little flow to the river. The report should discuss the impact of reducing K in Layer 1.

Response:

The sensitivity analysis, presented in the response to General Comment 1, indicates that the model is relatively insensitive to moderate changes in Layer 1 hydraulic conductivity. An increase in shallow layer transmissivity results in a flow increase of less than one gpm. Therefore, varying the hydraulic conductivity of Layer 1 to obtain better modeling results is considered appropriate.

Also, data available for the Discharge Control Study (Attachment 1) suggested that a lower hydraulic conductivity for Layer 1 was appropriate. First, Site R geologic cross-sections (Attachment 4) indicated that the shallow layer was comprised primarily of clay. Second, studies performed by Geraghty and Miller (Attachments 4 and 5)) showed a range of slug test values from $9x10^{-5}$ cm/sec to $6x10^{-3}$ cm/sec for the shallow layer and characterized it as being a "low permeability zone with fine-grained silty sand deposits predominating." Third, a review of the large-scale cross section by Bergstrom and Walker (Attachment 7) shows the upper portion of the cross section being largely comprised of fine-grained material.

Comment No. 6 **Modeling Approach, Page 5, Paragraph 2**

This paragraph discusses determination of the flow rate of contaminated groundwater to river cells in Layers 1 and 2. The paragraph does not fully discuss the hydraulic and physical attributes of the river cells used in the model. To fully conceptualize the hydraulic and physical properties of the river cells, information such as the length of each river reach, the width of the river, and the thickness of the riverbed should be provided (McDonald and Harbaugh, 1988). The report should include this information.

Response:

The areal extent of each river cell is the same as the model grid shown in Figure 2 (Attachment 1). A riverbed conductance of 795 ft^2/day was used for a 60 ft by 60 ft cell, and proportionally higher conductances were used for cells with larger areas.

The following bottom elevation profile, derived from the fourth transect from the north on USACE River Mile Map 176.2 to 178.2 (Attachment 1) was used in the model:

Distance from Eastern Shore of	Measured Bottom Elevation in River

River (ft)	(ft msl)
0-60	385
60-120	380
120-180	378
180-240	375
240-300	372
300-1080	370
1080-1620	360
1620+	370

This bathymetry transect is aligned with the center of Site R.

Comment No. 7 **Modeling Approach, Page 5, Paragraph 2**

This paragraph discusses evaluation of different flow control pumping schemes. The report states that the "most vulnerable location for river flow inflow to a Site R flow control well was determined." The report does not explain what "vulnerable" means in the context of flow rates from the river and discharge rates from the pumping well. Also, the report does not discuss how the "most vulnerable location" was determined. The report should clarify these matters. In addition, the paragraph refers to a "critical well" at which the discharge rate was increased to determine the pumping rate that would cause inflow from the river. The report does not clearly state which well is the "critical well," where this well is screened, or where this well is located. The report should clarify these matters as well.

Response:

The "most vulnerable well" and "critical well" are defined as the well location that is closest to the river but immediately downgradient of the capped area on Site R. In other words, the pumping well located where the distance between the downgradient edge of Site R and the Mississippi River is the smallest. This is the well location most susceptible to recharge from river infiltration, which is a critical design constraint for the groundwater migration control system.

Comment No. 8 **Modeling Results, Page 5, Paragraph 2**

This paragraph states that the maximum sustainable pumping rate that does not result in inflow from the river is between 200 and 250 gpm. U.S. EPA assumes that this range is based on the discussion in the "Modeling Approach" section; however, the report does not clearly explain how the range of 200 to 250 gpm was determined. The report should explain the determination of these values.

Response:

"Significant" flow is defined as the maximum amount of water that can be pumped from a recovery well located between Site R and the Mississippi River without inducing recharge from the river. As shown on Figure 6 (Attachment 1), the shortest distance between the downgradient edge of Site R and the Mississippi River occurs approximately 500 feet north of the south end of Site R. A well pumping at the "significant" flow rate at this location has a limited impact on the equipotential line that is aligned with the eastern boundary of the Mississippi River. Qualitative review of this pumping scheme indicated that most of the flow into the well at this pumping rate was from upgradient portions of the water-bearing unit, and not from the river. Model runs with higher pumping rates showed that the equipotential line on the river was captured by the well, resulting in "significant" flow from the river to the well.

Comment No. 9 **Figure 1**

This figure does not contain a legend that defines the various colored zones in the modeled area. The figure should include a more complete legend that defines these zones. Also, this figure does not identify the locations of any confining layers in the saturated zone. The locations of any confining layers present should be identified in the figure.

Response:

A revised Figure 1 is included in Attachment 1. The dark blue color is the Mississippi River, light blue are the middle and deep layers, light green is the shallow horizon, yellow is the unsaturated zone, and grey is bedrock. A note has been included indicating that the shallow layer serves as a semi-confining unit.

Comment No. 10 **Figure 5**

This figure depicts the locations of river discharge zones and discharge control wells for the three modeled horizons in the study area. Layers 2 and 3 appear to be mislabeled in the figure.

Response:

A corrected Figure 5 is included in Attachment 1.

Comment No. 11 **Attachment C**

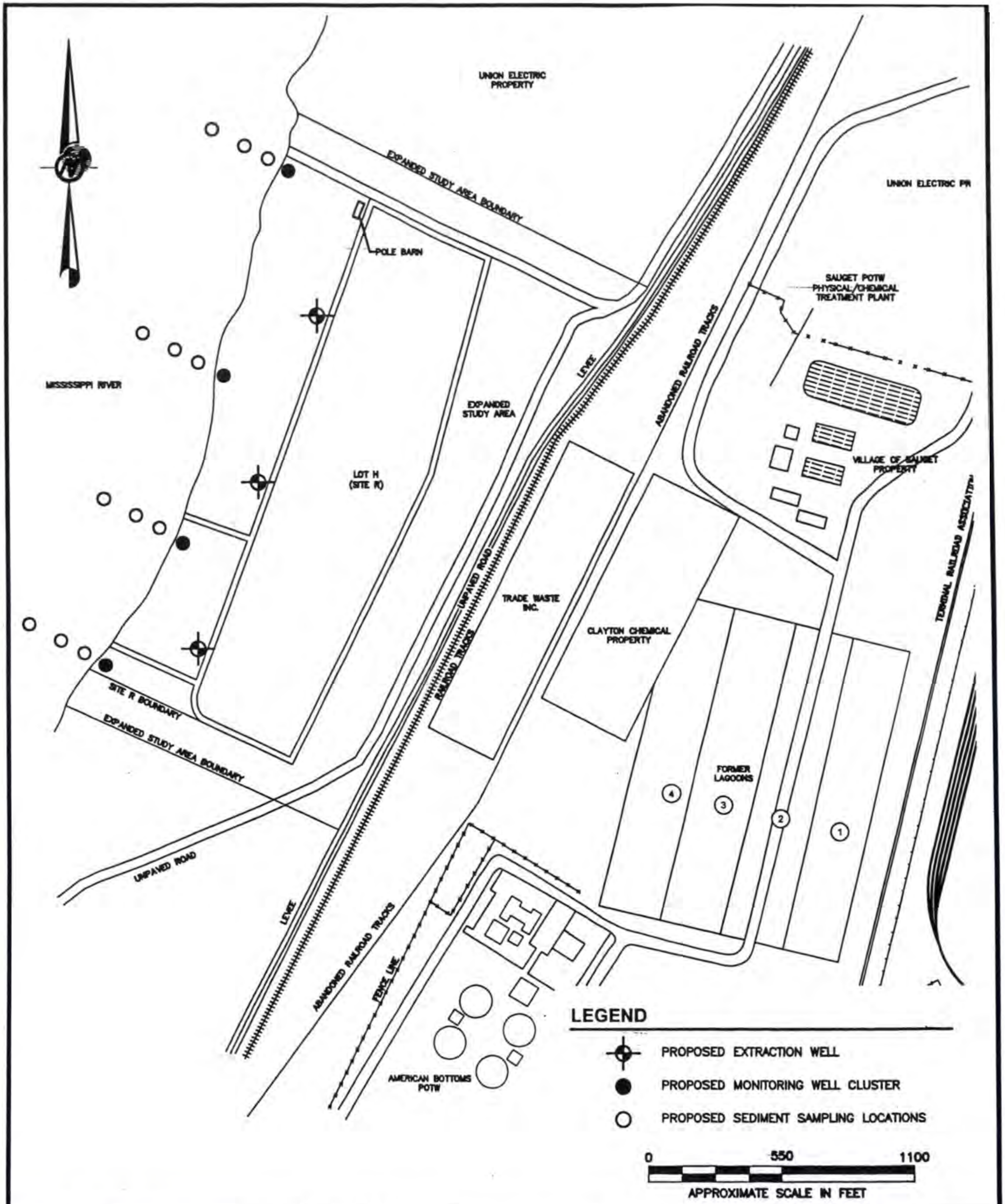
This attachment contains a U.S. Army Corps of Engineers (U.S. ACE) map of the Mississippi River (river miles 178.2 to 180.3) and is represented as being in the vicinity of Site R. However, a more appropriate U.S. ACE river map containing Site R— that for river miles 176.2 to 178.2— is not included. The attachment should include the U.S. ACE river map containing Site R (river miles 176.2 to 178.2) and the appropriate hydrographic data from this map used as input in the MODFLOW model.

Response:

The wrong map was included in Attachment C. The correct map is included in Attachment 1. The fourth transect from the left (north) on the map, which aligns with the center of Site R, was used to delineate the river bottom elevations for the Discharge Control Study (Attachment 1).

Figure 1

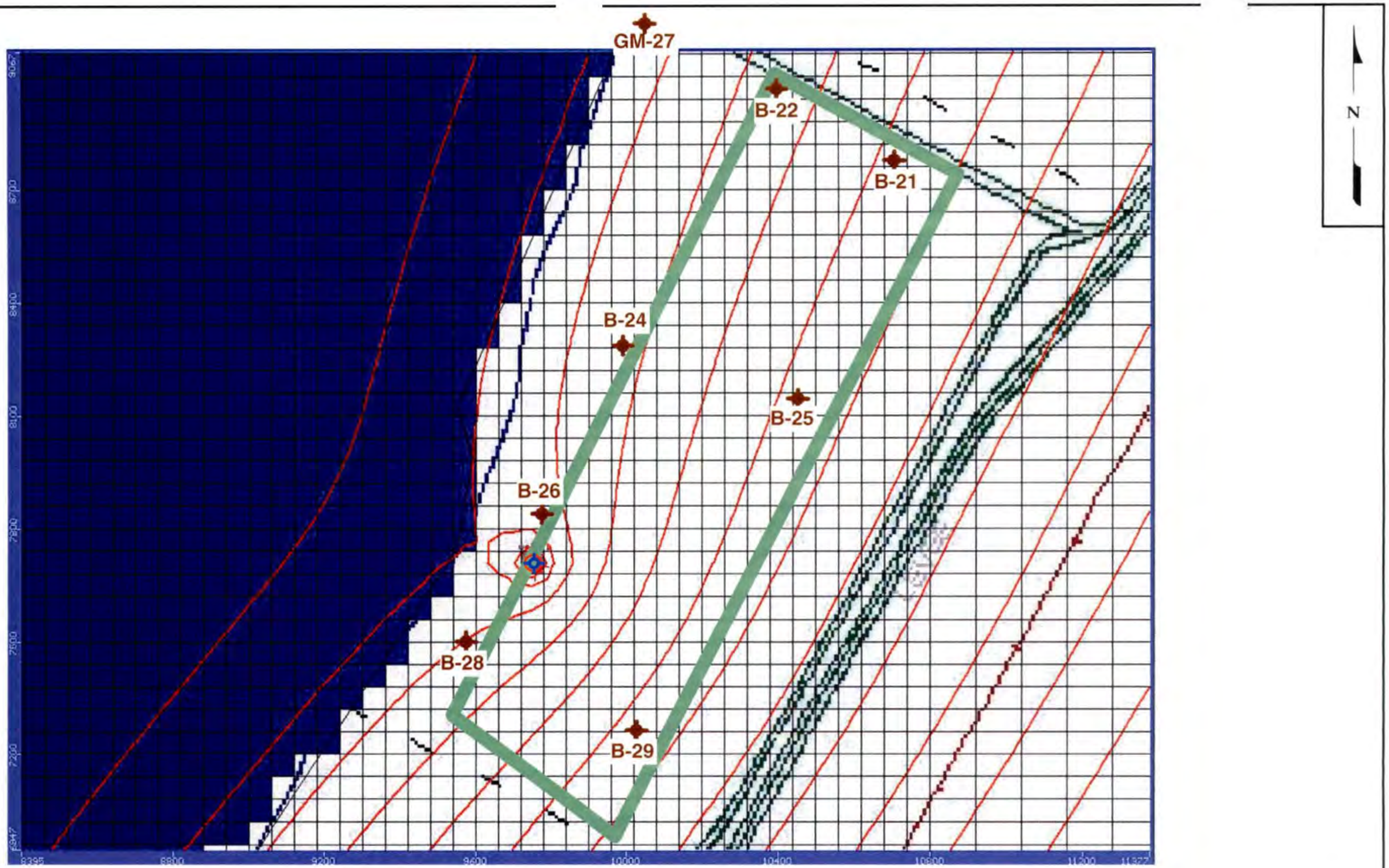
Proposed Monitoring Plan



Golder Associates St. Louis, Missouri	SCALE AS SHOWN	TITLE	
	DATE DEC 2001	PROPOSED EXTRACTION WELL LOCATIONS GROUNDWATER ALTERNATIVE B - HYDRAULIC BARRIER	
	DESIGN DESIGN		
	CADD MSL	PROJECT FOCUSED FEASIBILITY STUDY INTERIM GROUNDWATER REMEDY Sauget Area 2, Sauget, Illinois	DWG/FIG No. 5-2
FILE No. 9665.5-2	CHECK JRS		
PROJECT No. 013-9665 REV. 2	REVIEW JRS		

Figure 2

Water-Level Measurement Wells



LEGEND

- Discharge control well
- Monitoring well cluster
- River cells in MODFLOW model
- Approximate area of Site R
- Head equipotential contour (ft)

PRELIMINARY

NOTES:
1) Grid size in Site R = 60 ft x 60 ft
2) River elevation is 291 ft MSL

SCALE (ft)
0 200 400

GROUNDWATER SERVICES, INC.

MONITORING WELL LOCATIONS DISCHARGE CONTROL STUDY

Site R, W.G. Krummrich Plant, Sauget, Illinois
Solutia Inc.

GSI Job No: G-2561		Drawn By: CRW	
Issued: 11/30/01		Chk'd By: SKF	
Revised: 12/21/02		Appv'd By: CJN	
Scale: As Shown		**FIGURE 2**	

Specific Comments

Comment No. 1 **Page 7, Section 2.2.1, Well Casing Pipe**

The text states that well casing pipe must be "10-inch I.D. low carbon stainless steel." However, the type of stainless steel and the thickness of the casing are not specified. Typically, specifications include this type of information. Low-carbon stainless steel is usually Type 316L. Also, the diameter of stainless steel pipe is usually identified in terms of outside diameter. The text should include this missing information.

Response:

The Specifications will call for stainless steel well casing, Type 304. In addition, the screen will be specified as Type 304 stainless steel. 10-inch, stainless steel casing size is generally specified in terms of inside diameter.

Section 2.2.1, Part A of the Specifications will be revised to read:

A. 10-inch I.D. Type 304 stainless steel pipe with flush threaded joints and Teflon "O" rings.

Comment No. 2 **Page 7, Section 2.2.2, Grout, Part A**

The text states that "neat cement grout" consists of "cement and water in proportion of 1 bag (94 lb) Portland cement to 8.3 gal clean water." Part B, however, states that the mix design "shall be approved by the REMEDIAL DESIGNER." It is not clear why the remedial designer has to approve the mix design when it is presented in Part A. Also, the type of cement required is not clear. The text should identify the type of cement required for the grout and clarify the mix design requirements.

Response:

The language in Part B is intended to provide some quality control capacity into the grout mixing process. The cement grout will be mixed in the proportions specified in Part A of Section 2.2.2

in the Specifications. The Specifications will call for Portland Type 1 cement. There will be no change to Part B of Section 2.2.2.

Section 2.2.2, Part A of the Specifications will be revised to read:

A. Neat Cement Grout: Cement and water in proportion of 1 bag (94 lb) Type 1 Portland cement to 8.3 gal clean water. Slurry weight of 13.4 to 14.5 lbs/gal. A mud balance shall be used to verify slurry weight.

Comment No. 3 **Page 8, Section 2.2.3, Screen, Part B**

The text specifies that the screen must be Type 304 stainless steel with a nominal diameter of 10 inches. Because low-carbon stainless steel is specified for the well casing pipe, it is advisable to use the same type of stainless steel for the well screen. Type 304 stainless steel has a carbon content of 0.08 percent, which is almost three times greater than the carbon content of the stainless steel specified for the well casing pipe. The material requirements for the screen should be reviewed in light of this information.

Response:

The Specifications will call for Type 304 stainless steel casing and screen. There will be no change to Section 2.2.3, Part B

Comment No. 4 **Page 9, Section 2.2.6, Part C**

The text calls for a "steel pitless case of the same size as [the] well casing, with black corrosion resistant coating." It is not clear why a corrosion-resistant coating is required; if stainless steel is used, corrosion should not be a significant problem. Also, it is not clear what type of "black corrosion resistant coating" is required. In addition, the material of construction for the pitless adapter is not specified. The text should clarify these matters.

Response:

Corrosion resistant coatings are standard on many pitless adapters and serve to prolong the life of the units. The Specifications will be altered to call for a steel pitless adapter, which will have an epoxy-based corrosion resistant coating, along with stainless steel seating rings.

Section 2.2.6, Part C of the Specifications will be revised to read:

C. Design Requirements:

1. Steel pitless case of same size as well casing, with epoxy corrosion resistant coating.
2. Drop pipe: 3-inch I.D. threaded connection.
3. Discharge connection: 3-inch threaded connection, 150 psi working pressure, depending on information from conveyance design.
4. Sealed conduit connection, with neoprene electrical cable seal and O-rings.
5. Lifting lugs.
6. Designed for stresses that may occur during installation, testing, and operation.
7. Stainless steel seating rings

Comment No. 5 **Page 9, Section 2.2.7, Galvanized Steel Drop Tubing**

It is not clear why galvanized steel tubing is specified for what appears to be a well pump discharge. Also, the thickness of the tubing is not specified. If the installation is designed for 30 years of useful life, stainless-steel, schedule 40 pipe should be specified as the discharge pipe material. Typically the well pump is supported by the discharge piping, as it is the pump's and piping's weight that maintains the seal in the pitless adapter. The text should be revised in light of this information and the construction material and the class or schedule of the discharge piping should be specified.

Response:

The Specifications will call for a Type 304 stainless steel drop pipe (well pump discharge pipe). It is advisable to support the weight of the pump with a safety chain or cable in addition to the riser pipe rather than by the pipe alone. This typically results lower stress on the pipe connections and results in a lower failure rate.

Section 2.2.7, Part A of the Specifications will be revised to read:

A. Design requirements:

1. 3-inch I.D. Type 304 stainless steel tubing.
2. Drop tubing shall connect to a stainless steel barb at bottom of pitless adapter and top of extraction well pump.
3. Tubing shall not support weight of pump. Pump shall be supported by a stainless steel chain attached to top of well or pitless adapter as appropriate.

Comment No. 6 **Page 10, Section 2.3.3, Part B**

Phrasing such as "it is suggested that" should be avoided. The specifications should be clear and concise regarding the drilling method to be used for installation of wells. If necessary, the specifications should include a provision for the contractor to propose an alternative drilling method that can be accepted or rejected by the engineer.

Response:

Section 2.3.3, Part B of the Specifications will be revised to read:

B. The extraction wells will be constructed using cable tool methods. However, in the event that drilling by cable tool is determined to be not feasible, another drilling method may be substituted. Drilling Methods shall be approved by the REMEDIAL DESIGNER and shall conform to all State and local standards for piezometer/well construction.

1. Acceptable drilling fluids are potable water and air.
2. Extraction wells shall be drilled straight and plumb.

Comment No. 7 **Page 10, Section 2.3.4, Part B**

The text implies that boreholes will be sampled during drilling; however, the sampling intervals and method are not specified. The text should clearly state the sampling intervals and procedures required.

Response:

Section 2.3.4, Part B of the Specifications will be revised to read:

B. Following drilling and sampling of the 16-inch diameter boreholes, install a 10-inch diameter screen concentrically in the open hole. Screen lengths specified to be 65 feet as shown on the Contract Drawings. Actual screen lengths shall be determined by the REMEDIAL DESIGNER based on the field conditions encountered. Place five feet of 10-inch diameter casing below the screen as a sand trap. The boreholes will be sampled for lithologic classification purposes. Continuous sampling will be accomplished by collecting small portions of the cable tool cuttings, for inspection and classification.

Comment No. 8 **Page 11, Section 2.3.5, Part H**

The text requires the contractor to conduct a short- duration performance test for each well. It is not clear what the objective of this test is or what will determine acceptance or rejection of a well. The text should be clearly state the test objective and the criteria for well acceptance.

Response:

Section 2.3.5, Part H of the Specifications will be revised to read:

H. Conduct a short-duration performance test on each well, initially using a flow rate of approximately 50 gpm. Increase the flow rate to 217 gpm for up to 1 hour, to verify performance. The performance tests will be conducted immediately after well installation if the discharge control system is tied into the Village of Sauget sewer system. If the system is not tied in, performance tests will not be conducted until such a tie in is made and the appropriate permits are obtained. The short-duration performance test will be performed in order to verify that the system is functioning per design specifications.

Comment No. 9 **Page 11, Section 2.3.6, Part A**

This section calls for decontamination of drilling equipment when it arrives on site and before it leaves the site. It is not clear, however, whether downhole drilling equipment will be decontaminated between boreholes. The text should clarify this matter.

Response:

While it is important to decon the drilling equipment before it arrives at the site and when it leaves the site, it will not be decontaminated between boreholes at the site. Recovery wells are located in areas of impacted soil and groundwater. Since cable tool equipment will be used to install the recovery wells, there is little opportunity for carry over of contaminants from one borehole to another because of the limited amount of impact that will contact impacted groundwater. Even if carryover occurs, there is no adverse consequence. These are not groundwater monitoring wells. They are groundwater recovery wells that will be pumping impacted groundwater with concentrations in the 10 to 100 ppm range.

Comment No. 10 **Page 12, Section 2.3.7, Part A**

The text discusses collection and containerization of liquids generated during well installation. However, it is not clear whether these liquids will be sampled for analysis or how they will be disposed of. The text should clarify this matter.

Response:

Section 2.3.7, Part A of the Specifications will be altered to read as follows:

A. All aqueous and non-aqueous liquids used, collected or encountered during the performance of this Work, including well development water and decontamination water, shall be collected in drums, delivered to a consolidation area and transferred to an appropriate bulk container, e.g. a double-walled tank or contained single-wall tank. Containerized liquids will be characterized to determine the proper disposal method and disposed off-site at a facility permitted to handle these liquid in accordance with applicable rules and regulations.

Comment No. 11 **Page 14, Section 3.1.3, Part B, Subpart 1**

The specification requires submittal of a "pump manufacturer's statement of overall efficiency guarantee for [the] pumping unit under specified conditions." The conditions are not clearly specified. The text should specify the conditions if such a guarantee is to be required.

Response:

Section 3.1.3, Part B, Subpart 1 of the Specifications will be altered to read as follows:

1. Pump manufacturer's statement of overall efficiency guarantee for pumping unit, under the conditions specified in Section 3.2.2.

Comment No. 12 **Page 14, Section 3.1.6, Parts A, B, and C**

If it is expected that pumping conditions will vary greatly, it would be prudent to specify pumps with variable-frequency drives. Such pumps would accommodate a range of flow rates and any future adjustments required by fluctuations in groundwater levels caused by pumping or seasonal factors. Dropping groundwater levels will increase the static head, reducing the pumping rate required. Pumps with variable-frequency drives can provide the desired discharge rate regardless of changes in static head. The text should be reviewed in light of this information and revised as necessary.

Response:

Variable frequency drives, although not considered necessary because valves can be used to control pump discharge rate, may be beneficial. Therefore, variable frequency drives will be specified in Section 3.2.2 of the Specifications. No change to Section 3.1.6 is required.

Section 3.2.2, Part B of the Specifications will read:

B. Provide submersible well pumping equipment in accordance with the requirements shown on the Contract Drawings, or as otherwise specified by the REMEDIAL DESIGNER. Pumps will utilize variable frequency drives, which are compatible with each pump, as recommended by the pump manufacturer.

Comment No. 13 **Page 15, Section 3.2.2, Part D**

The specification states that "pumps shall be sized to provide at least 220 gallons per minute (plus or minus 20%) flow rate against a total head of at least 70 feet, depending on final design parameters for the conveyance system." This is an unusual requirement. Typically, pump specifications state the required pumping rate at a fixed total dynamic head for constant-speed pumps. The plus or minus 20 percent allowance for the flow rate might allow the contractor to choose between two pump sizes, and the contractor would probably furnish the smaller or less expensive pump, which might be too small in the long run. The text should be reviewed based on these considerations and revised as necessary.

Response:

Section 3.2.2, Part D of the Specifications will be altered to read as follows:

C. Pumps shall be sized to provide at least 220 gallons per minute (plus 20%) flow rate against a total head of at least 70 feet, depending on final design parameters for the conveyance system.

Comment No. 14 **Page 15, Section 3.2.3**

According to Drawing No. 3, it appears that the check valve will be located in each well on top of the well pump. This placement of the check valve may cause maintenance problems, as the valve would not be accessible. Also, high shutoff head may cause the valve to fail prematurely as a result of flow reversal. Because the drawings indicate use of valve vaults, it may be advisable to install the check valves in these vaults, where they will be easily accessible for maintenance and will not be exposed to high shutoff head. Additionally, check valves for such installations are usually specified to be of stainless-steel construction. The design should be reviewed in light of these considerations, and the specification should be revised as necessary.

Response:

Placement of the check valves in vault will cause backflow through the pumps in the event of pump shutdown. This backflow may shorten the life of the pump particularly if the pump is energized while water is flowing down the drop pipe and through the pump impellers. Therefore, the check valves are placed near the pumps. The Specifications will call for stainless steel check valves.

Section 3.2.3, Part C of the Specifications will read:

C. Stainless steel construction, stainless steel spring, threaded ends, double disc type.

Comment No. 15 **Page 16, Section 3.2.4, Part B**

The text requires the contractor to "provide flow control valves to prevent flow rates above [the] operating range of [the] well pump." However, it is not clear what this operating range is. It is also not clear what types of valves are required or what their materials of construction are to be. Use of pumps equipped with variable-frequency drives would eliminate the need for these valves (see comment no. 12). The design should be reviewed in light of these considerations, and the specification should be revised as necessary.

Response:

As noted in the response to Comment 12, variable frequency drives will be installed for each well. In addition, Section 3.2.4, Part B of the Specifications will be altered to read as follows:

B. Provide steel, 3-inch ball valves as depicted on Drawing No. 4, so that each well can be fully isolated (for example, during repairs).

Comment No. 16 **Page 17, Section 3.2.7, Part E**

The text should be revised to read as follows: "Pump motors shall be non-overloading throughout their entire operating range."

Response:

Section 3.2.7, Part E of the Specifications will be revised to read:

E. Pump motors shall be non-overloading throughout their entire operating range.

Comment No. 17 **Page 27, Section 5.2.2, Part B**

The text states that the pumps will be operated by level switches located in each extraction well. However, the operating range of the switches and the distance between the switches in each well are not specified. Also, it is not clear whether fluctuations in groundwater levels will be addressed by the control scheme. These matters should be clarified. In addition, the text indicates that a high high-level switch will initiate a remote alarm in the Department 277 control room, which will be more than 6,000 feet away from one extraction well. The remote alarm is not shown on the drawings, and therefore it is not clear whether this alarm will be hard-wired or activated via an autodialer. The text and drawings should be revised to clarify this matter.

Response:

The distances between the switches are not specified because final aquifer well parameters are unknown and well specific capacity has not been determined. These distances will be set based on the performance test specified in Section 2.3.5 of the Specifications. The Specification text and drawings will clarify that the remote alarm, if utilized, will be hard-wired to the control panel. Revised drawings are included in Attachment 10 Hydraulic Barrier Design Drawings. Drawing No. 3 of Attachment 10 was revised to reflect this change.

Section 5.2.2, Part B of the Specifications will read:

A. Level Switches / Alarms

The pumps in each extraction well shall be controlled primarily by the level switches. There will be three level switches per extraction well; a low-level switch (LSL), high-level switch (LSH), and a high-high level switch (LSHH). The level switches shall be adjustable. Setpoints shall be determined by the REMEDIAL DESIGNER for each extraction well, based on the results of the performance test specified in Section 2.3.5. of the Specifications.

In automatic mode, the low-level switch shall turn off the pump. No alarm shall exist when the groundwater level reaches this level. When the groundwater rises above the high-level switch, the level switch shall energize a relay in the control panel to turn the pump on and illuminate a light on the control panel indicating the pump is running.

The high-high level switch will be used to indicate a high level condition. If groundwater level passes the high high-level switch, the level switch shall energize a high-level alarm relay in the control panel to Illuminate the high level alarm light on the control panel.

A motor overload in the motor starter will be used to indicate a pump failure. If a pump motor overloads, a switch shall activate a pump failure alarm relay in the control panel to Illuminate the pump failure alarm light on the control panel.

Comment No. 18 **Drawing No. 2**

This drawing shows what appears to be a single force main to which the three well pump discharges will be connected. It is difficult to evaluate this system because no pipe sizes

are included on the drawings or in the specifications. This information should be shown on the drawings to facilitate the review process and avoid confusion during bidding. Drawing No. 2 also shows electrical lines to be underground electrical feeders. However, the conductor sizes required are not shown. This information should be provided on the drawing. It should be noted that running a long feeder from the pole barn to the well located furthest south will likely produce a voltage drop because of the distance involved (about 2,000 feet).

Response:

Drawing No. 2 will call for electrical feed lines to consist of No. 6 AWG copper wire, or other wire specified by a licensed professional electrician. Drawing No. 2 will call for the force main to consist of 10-inch diameter, high density polyethylene (HDPE) pipe, pending final design of the conveyance system. Revised drawings are included in Attachment 10.

Comment No. 19 **Drawing No. 3**

The specification in Section 3.2.6 calls for a submersible cable with at least 5 extra feet available for termination in a junction box at the "pump head, or wellhead." Drawing No. 3 does not indicate that the submersible cable will terminate in a junction box at the wellhead. Rather, the drawing indicates that the submersible cable will enter the electrical conduit through the well casing and run underground to a junction box just below the control panel; this is the only junction box shown. The discrepancy between the text and drawing should be reconciled. The drawing also indicates that power cables and flow sensor telemetry will be installed in the same schedule 40 polyvinyl chloride conduit. Installing power and telemetry wiring in one conduit is not recommended. This design element should be reviewed and revised as necessary.

Response:

Drawing No. 3 will be altered to indicate a junction box located at the wellhead. Drawing No. 3 is meant to indicate that telemetry and power wiring will be run in two separate conduits, and will be clarified to better convey this information. Revised drawings are included in Attachment 10.

Comment No. 20 **Drawing No. 4**

It is not clear why manhole steps are required in the vault shown. The vault will be only 3.5 feet deep and will have a sampling port located under the 30-inch-square access cover, making it impossible to enter. Also, the 3-inch ball valve downstream from the sampling line will be difficult to operate as it is presently configured. In addition, flow sensor wiring should be terminated in a waterproof junction box. Moreover, a provision such as a French drain should be included in the design to remove accumulated rainwater. The drawing should be reviewed in light of these considerations and revised as necessary.

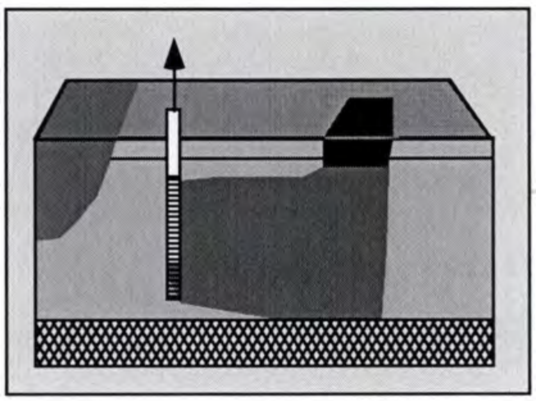
Response:

The manhole steps shown in Drawing No. 4 will be removed. The 3-inch ball valve will be moved underneath the access cover. The flow sensor wiring will be terminated in a waterproof junction box. The pipe and one-way valve shown in profile view A-A' is intended to drain the vault. A French drain may allow water to enter the vault during times of high river stage. Revised drawings are included in Attachment 10.

DISCHARGE CONTROL STUDY

Site R
W.G. Krummrich Plant

Sauget, Illinois



PRELIMINARY

Submitted to
Solutia Inc.

Nov. 29, 2001

Groundwater Services, Inc.
2211 Norfolk, Suite 1000, Houston, Texas 77098

EXECUTIVE SUMMARY

Site R is located in the American Bottoms area on the east bank of the Mississippi River and west of the W.G. Krummrich Plant. In this report, Site R refers to a capped area approximately 2000 ft wide (perpendicular to groundwater flow) and 500 long (parallel to groundwater flow). Below Site R, affected groundwater extends from close to the water table to bedrock (typically from 30 ft to 140 ft below ground surface).

The first objective of this study was to estimate the flowrate of affected groundwater from the water-bearing units underlying Site R to the Mississippi River during average river level conditions. The second objective was to determine an efficient pumping scheme for extracting groundwater at this flowrate without causing inflow of Mississippi River water to the flow control wells.

Results

A numerical groundwater flow model, MODFLOW, was used to meet these objectives (Figures 1-5). The modeling analysis indicated that the flowrate of affected groundwater from the water-bearing units underlying Site R to the Mississippi River during average river level conditions is **650 gpm**.

The maximum pumping rate for a well downgradient of Site R that can be sustained without any inflow from the river is between 200 and 250 gpm. This indicates that **three wells, each pumping 217 gpm,** would result in a total flowrate of 650 gpm without significant inflow from the Mississippi River.

INTRODUCTION

As requested by Solutia Inc. (Solutia), Groundwater Services, Inc. (GSI), has completed a design basis study for affected groundwater associated with Site R near the W.G. Krummrich Plant in Sauget, Illinois. This report summarizes the approach and results of the study.

PROJECT BACKGROUND

Site R is located in an area referred to as the American Bottoms on the east bank of the Mississippi River directly downgradient of the W.G. Krummrich Plant. The geology of the area is described as consisting of unconsolidated valley fill deposits (Cahokia Alluvium) overlying glacial outwash material (Henry Formation). In general, the permeability of the unconsolidated material increases with depth with the outwash material being comprised of medium- to coarse-grained sand and gravel. The hydrogeologic conceptual model divides the unconsolidated water-bearing unit into three horizons: the shallow horizon (extending 400 to 380 ft MSL), the middle horizon (extending from 380 to 350 ft MSL) and the deep horizon (extending from 350 ft MSL to bedrock, or about 290 ft MSL at Site R).

In this report, Site R refers to a capped area approximately 2000 ft wide (perpendicular to groundwater flow) and 500 ft long (parallel to groundwater flow).

Representative constituents associated with Site R include volatile organic constituents (VOCs) such as benzene, chlorobenzene, acetone, and 1,2-dichloroethane and semi-volatile organic constituents (SVOCs) such as phenol, 2-chloroaniline, and 2-nitrochlorobenzene. Site constituents are found from the water table to bedrock in all three horizons.

The first objective of this study was to estimate the flowrate of affected groundwater from the water-bearing units underlying Site R to the Mississippi River during average river level conditions. The second objective was to determine an efficient pumping scheme for extracting groundwater at this flowrate without causing inflow of Mississippi River water to the flow control wells. A numerical groundwater flow model, MODFLOW, was used to meet these objectives.

MODEL DESCRIPTION

The MODFLOW groundwater flow model, developed by the U.S. Geological Survey (McDonald and Harbaugh, 1988), was used to simulate the movement of groundwater for baseline conditions and for various pumping scenarios.

Key MODFLOW Model Attributes, Assumptions, and Input Parameters

Key model attributes and assumptions for the MODFLOW model are listed below:

- A finite-difference grid with 60 ft by 60 ft cells in the vicinity of Site R was used with cell size gradually increasing with distance from Site R (Figure 1). Adjacent model cell column and row widths were not altered more than a factor of 2.0 compared to adjacent columns (Zheng and Bennett, 1995 and Spitz and Moreno, 1996). The grid aspect ratio (ratio of column width to row width) was limited between 10 and 0.1.

- Three layers were used in the model: an unconfined shallow layer, a convertible confined/unconfined middle layer, and a confined deep layer. (The pumping regimes never resulted in unconfined conditions in the middle layer). The top and bottom elevations of the water-bearing units were derived from geologic cross-sections developed by IT Corporation (2001), Geraghty and Miller (date unknown), and Bergstrom and Walker (1956) (Appendix A).

- Hydraulic conductivity data compiled by Schicht (1965) was used as the initial hydraulic conductivity in the model for the middle and deep horizons (Figure 2).

- Bedrock elevations, obtained by kriging data contained in Bergstrom and Walker (1956) (Appendix B), were imported into the model.

- Corps of Engineers hydrographic data of the Mississippi River (COE, 2000) near Site R was used as input for a series of river cells in the MODFLOW model to simulate the configuration of the model (Figure 1 and Appendix C). The bathymetry of the river adjacent to Site R was assumed to extend throughout the entire model reach.

- An average river level stage of 391 ft MSL was used for the river in the study area based on 1933 to 2001 monthly river stage data (Appendix D).

- The riverbed conductance was assumed to be 3180 ft^2/day based on data developed by Schicht (1965).

- Constant head cells were used in the model to represent the eastern boundary of the modeled area (the bluff line) based on "steady-state" constant head elevations used in a regional groundwater flow model developed by Clark (1997).

- A surface infiltration rate of 7.8 inches per year was used in the model to represent infiltration from rainfall (Schicht, 1965).

- A regional pumping center of 4167 gpm was established in the model to represent ongoing highway dewatering projects in the East St. Louis area (Ritchey and Schicht, 1982).

Modflow Calibration

Flow calibration against water levels measured on October 25, 2001 was performed by adjusting the river level to 398.5 ft (the average river level for the 24 hrs preceding the midpoint of the sampling period) and comparing the predicted values to the actual modeled values (Table 1). Based on the calibration process, the following changes were made to the model:

- The hydraulic conductivity near the site was increased from the Schicht (1965) values in order to reduce the head difference between the river and near-river heads in Layer 2 and Layer 3 (see Figure 3 for final hydraulic conductivity values).

- The horizontal hydraulic conductivity of Layer 1 (K_x and K_y) was reduced from 0.01 cm/sec to 0.0005 cm/sec, and the vertical hydraulic conductivity was decreased to $1x10^{-6}$ cm/sec to better match observed hydraulic heads. Note that even with these changes, the match in Layer 1 was not as good as the Layer 2-3 match. Because of the small contribution to flow to the river from Layer 1, this match was considered to be acceptable.

In general, the potentiometric surface from the middle horizon was compared to the potentiometric surface for November 1990 reported by Schicht and Buck (1995). This comparison indicated a good relative match, as the general shape and values of the predicted potentiometric surface were similar to the reported potentiometric surface (including the cone of depression caused by the highway dewatering system). (The predicted values did not provide an absolute match to the observed values due to differences in river stage). Overall, the MODFLOW groundwater flow model was considered to yield a reasonable simulation of the aquifer system.

Modeling Approach

ZoneBudget is a water balance component of the Visual MODFLOW package that reports the exchange of groundwater between adjacent zones that are set up by the user. To calculate the quantity of groundwater discharge to the river, river cells downgradient of Site R were assigned into two zones, one for river cells in Layer 1 and one for river cells in Layer 2 (there were no river cells in Layer 3). This represented an area 2000 ft long parallel to the riverbank and extending all the way across the river. Then, by using ZoneBudget, the flowrate of affected groundwater to these zones during average flow conditions was determined.

To evaluate the efficiency of different flow control pumping schemes, a design goal of extraction without causing inflow of river water was applied. First, the most vulnerable location for river inflow to a Site R flow control well was determined. This was the point on the downgradient (western) side of Site R that is nearest to the river (Figure 6). Second, the MODFLOW model was used to determine the maximum pumping rate that could be achieved without causing inflow to the pumping well. This was achieved by increasing the pumping rate from the critical well until the potentiometric surface contours indicated that inflow from the river was occurring (Figure 6).

Modeling Results

The modeling analysis indicated that the flowrate of affected groundwater from the water-bearing units underlying Site R to the Mississippi River during average river level conditions is 650 gpm.

The maximum pumping rate that can be sustained without inflow from the river is between 200 and 250 gpm. This indicates that three wells, each pumping 217 gpm, would result in a total flowrate of 650 gpm without significant inflow from the Mississippi River.

KEY POINTS

- Flow from Site R to Mississippi River: **650 gpm.**
- Most efficient design for extracting 650 gpm without inflow from river: **3 wells pumping at 217 gpm each.**

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GSI Job No. G-2561
January 24, 2002

GROUNDWATER SERVICES, INC.

Discharge Control Study
Site R - W. G. Krummrich Plant
Sauget, Illlinois

Tables

GROUNDWATER SERVICES, INC.

Table 1
MODFLOW CALIBRATION RESULTS

Flow Discharge Study
Site R, Solutia Inc., W. G. Krummrich Plant, Sauget, Illinois

Well ID	Roux Elevation to Measuring Point (ft MSL)[1]	OBSERVED Depth to Water (Oct. 25, 2001) (ft)	OBSERVED Water Elevation (Oct. 25, 2001) (ft MSL)[2]	SIMULATED Water Elevation (ft MSL)[3]	Residual Error (SIMULATED-OBSERVED) (ft)	Squared Residual Errors (ft)
Layer 1						
B-22A	428.16	29.16	399.00	395.2	-3.80	14.44
B-24A	422.49	23.39	399.10	394.8	-4.30	18.49
B-25A	428.47	30.02	398.45	396.4	-2.05	4.20
B-26A	423.71	27.87	395.84	393.7	-2.14	4.58
B-28A	423.04	26.18	396.86	392.5	-4.36	19.01
B-29A	429.03	32.17	396.86	396.4	-0.46	0.21
				MEAN OF RESIDUAL ERRORS:	-2.85	
					ROOT MEAN SQUARE:	3.19

Well ID	Roux Elevation to Measuring Point (ft MSL)[1]	OBSERVED Depth to Water (Oct. 25, 2001) (ft)	OBSERVED Water Elevation (Oct. 25, 2001) (ft MSL)[2]	SIMULATED Water Elevation (ft MSL)[3]	Residual Error (SIMULATED-OBSERVED) (ft)	Squared Residual Errors (ft)
Layers 2 and 3						
B-21B	428.37	38.39	389.98	391.4	1.42	2.02
B-24C	422.52	32.80	389.72	390.7	0.98	0.96
B-25B	427.35	37.21	390.14	391.8	1.66	2.76
B-26B	423.62	33.58	390.04	390.7	0.66	0.44
B-28B	423.08	33.09	389.99	390.6	0.61	0.37
B-29B	429.06	38.83	390.23	391.5	1.27	1.61
GM-27B	426.04	36.09	389.95	390.9	0.95	0.90
GM-27C	426.76	36.63	390.13	390.9	0.77	0.59
				MEAN OF RESIDUAL ERRORS:	1.04	
					ROOT MEAN SQUARE:	1.10

NOTES:

1. Obtained from Table 2 of "Summary of Ground-Water Quality Conditions", Roux Associates, Inc., Vol. II of II, December 1997.
2. Calculated by GSI using elevations obtained from Table 2 of "Summary of Ground-Water Quality Conditions", Roux Associates, Inc., Vol. II of II, December 1997.
3. Groundwater elevations obtained from MODFLOW using a river elevation of 389.5 ft.
   ft = feet
   MSL = mean sea level

GSI Job No. G-2561
January 24, 2002

GROUNDWATER SERVICES, INC.

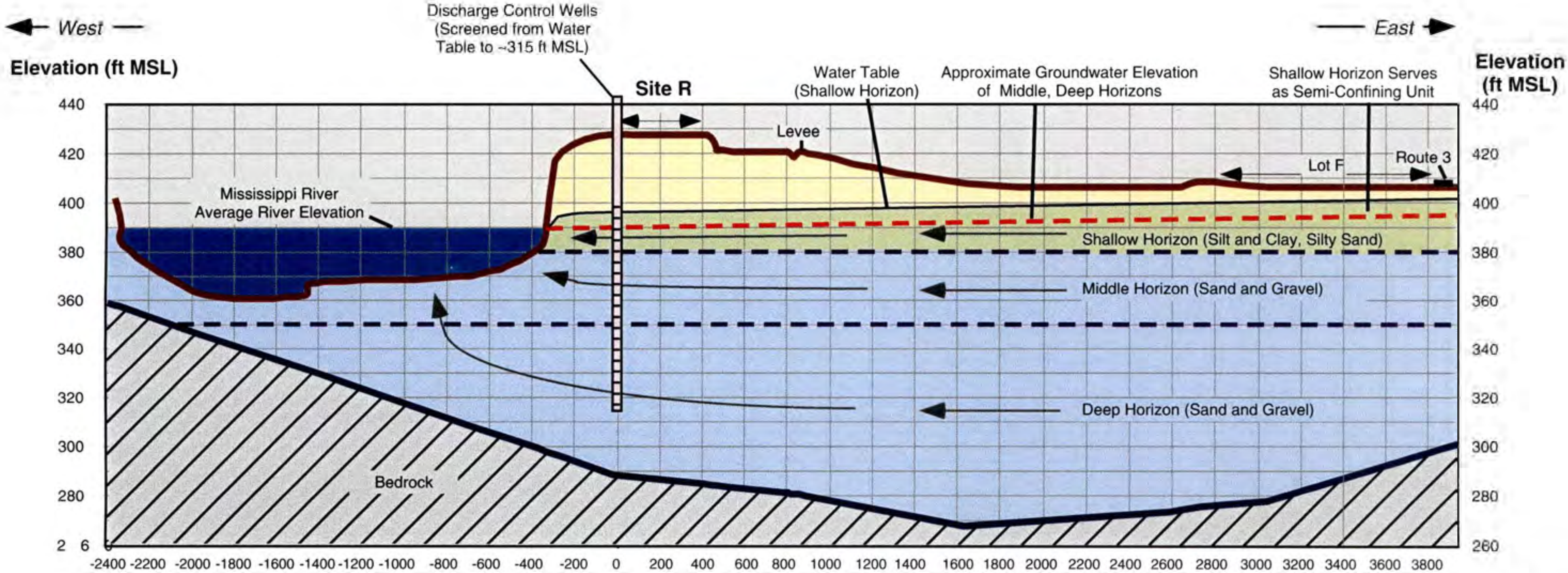
Discharge Control Study
Site R - W. G. Krummrich Plant
Sauget, Illlinois

Figures

GSI Job No.G-2561
Issued: 11/29/01
Revised: 1/21/02
PRELIMINARY

Figure 1
GENERALIZED HYDROGEOLOGIC CROSS SECTION

Discharge Control Study,
Solutia Inc., Site R, Krummrich Plant, Illinois



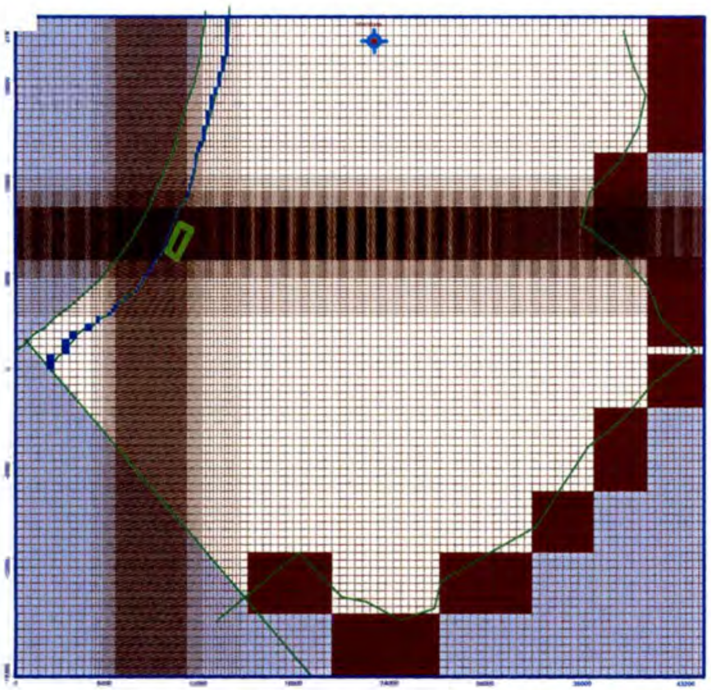
VERTICAL EXAGGERATION: APPROXIMATELY 12:1
VERTICAL SCALE: 1" = 66 FT
HORIZONTAL SCALE 1" = 752 FT

Sources:

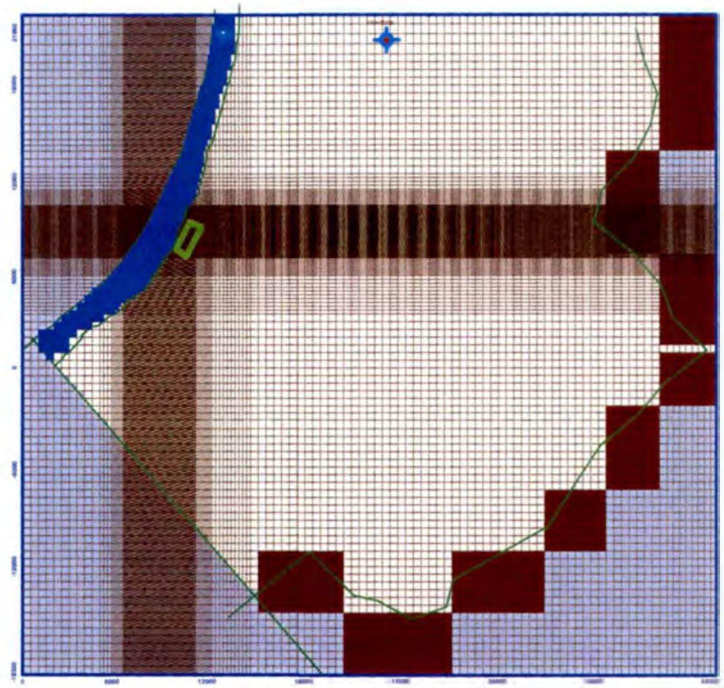
- Bedrock Elevation: Bergstrom and Walker, 1956
- Layer Elevations: URS Geologic Cross Section, 9/01, Geraghty and Miller, 1986, Groundwater Services, Inc., 2001
- Ground Surface Elevations: Geraghty and Miller, 1986 and URS Geologic Cross Section, 9/01
- Groundwater Flow Direction: Groundwater Services, Inc., 2001
- River Hydrography: 1994 Corps of Engineers data.

LEGEND

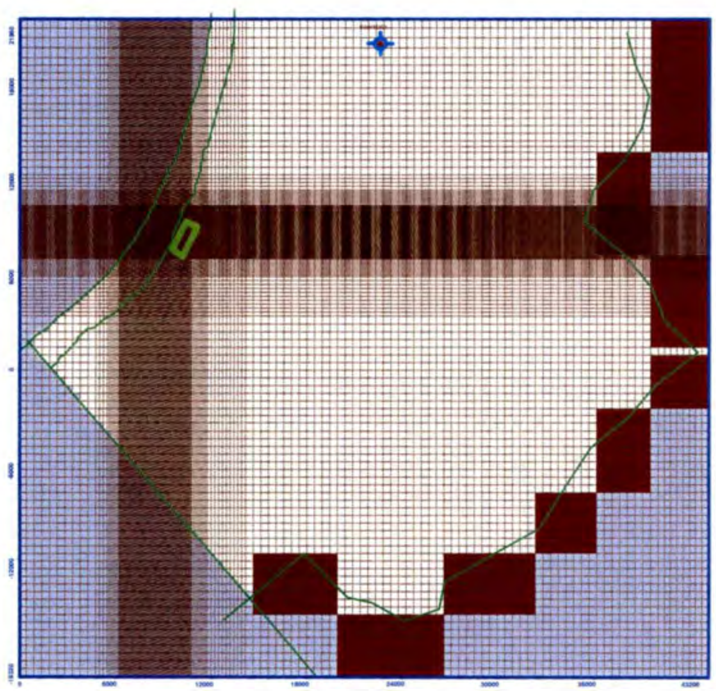
- *Approximate Direction of Groundwater Flow With No Pumping*
- *Mississippi River*
- *Unsaturated Zone*
- *Shallow Horizon in MODFLOW model*
- *Middle and Deep Horizons in MODFLOW model*
- *Bedrock*



Layer 1: Shallow Horizon



Layer 2: Middle Horizon



Layer 3: Deep Horizon

N

SCALE (ft)

0 6000 12,000

PRELIMINARY

LEGEND

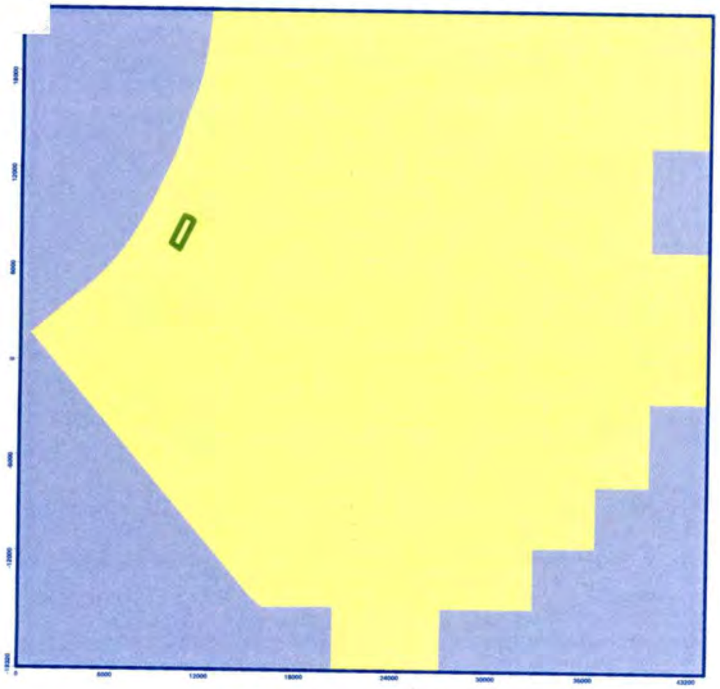
- Regional pumping center for highway dewatering
- River cells in MODFLOW model
- Approximate area of Site R
- Constant head cells
- Inactive cells

GROUNDWATER SERVICES, INC.

MODFLOW MODEL CONFIGURATION DISCHARGE CONTROL STUDY

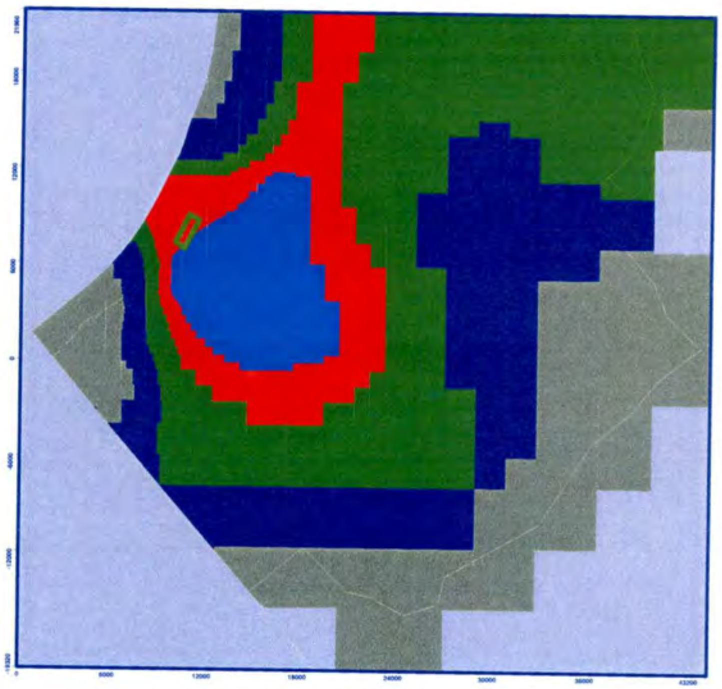
Site R, W.G. Krummrich Plant, Sauget, Illinois
Solutia Inc.

GSI Job No:	G-2561	Drawn By:	CRW
Issued:	11/30/01	Chk'd By:	SKF
Revised:	————	Appv'd By:	CJN
Scale:	As Shown		**FIGURE 2**



Layer 1: Shallow Horizon

Layer 2: Middle Horizon



Layer 3: Deep Horizon

N

SCALE (ft)

0 6000 12,000

LEGEND

	$K_x = K_y$ (cm/sec)	K_z (cm/sec)
Yellow	5 E-4	1 E-6
Gray	0.059	0.0295
Dark blue	0.083	0.0415
Green	0.11	0.055
Blue	0.15	0.075
Red	0.137	0.0685

Approximate area of Site R

Inactive cells

NOTE: K_x = Hydraulic conductivity in x direction
K_y = Hydraulic conductivity in y direction
K_z = Hydraulic conductivity in z direction

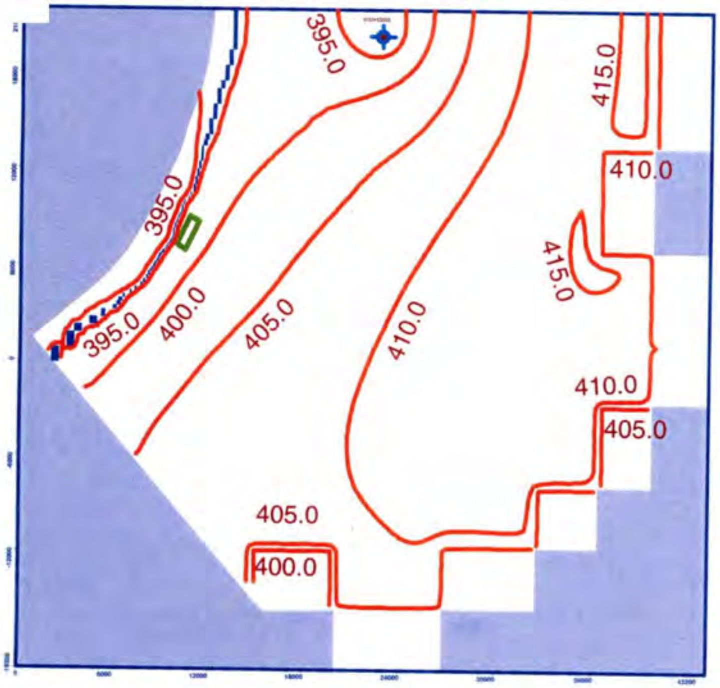
PRELIMINARY

GROUNDWATER SERVICES, INC.

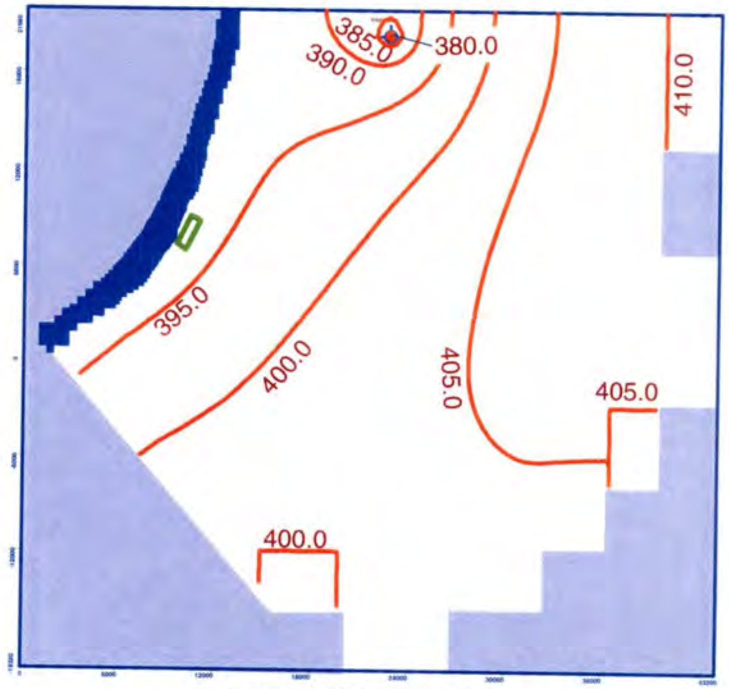
HYDRAULIC CONDUCTIVITY ARRAYS
DISCHARGE CONTROL STUDY

Site R, W.G. Krummrich Plant, Sauget, Illinois
Solutia Inc.

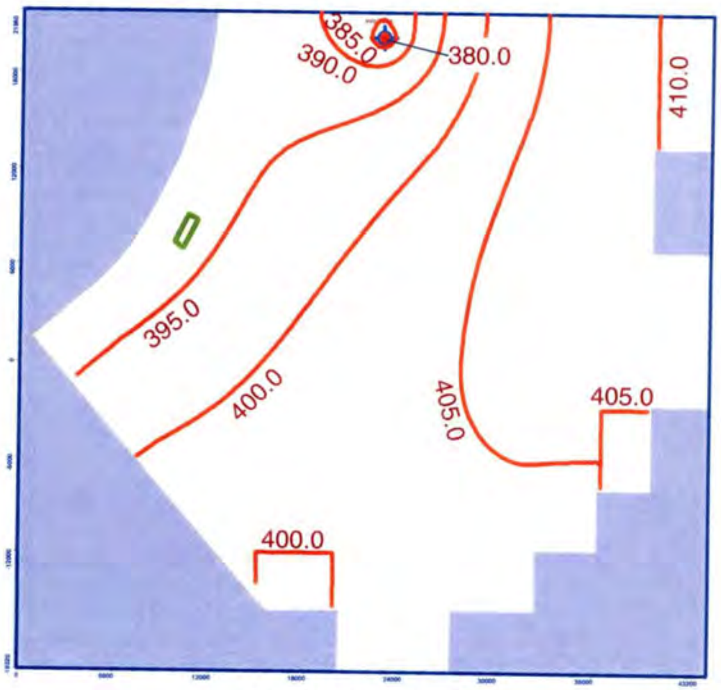
GSI Job No:	G-2561	Drawn By:	CRW
Issued:	11/30/01	Chk'd By:	SKF
Revised:	———	Appv'd By:	CJN
Scale:	As Shown		**FIGURE 3**



Layer 1: Shallow Horizon



Layer 2: Middle Horizon



Layer 3: Deep Horizon

PRELIMINARY

LEGEND

- Regional pumping center for highway dewatering
- River cells in MODFLOW model
- Inactive cells
- Approximate area of Site R
- Head equipotential contour (ft)

NOTE: 1. River elevation is 391 ft MSL.

SCALE (ft)

0 6000 12,000

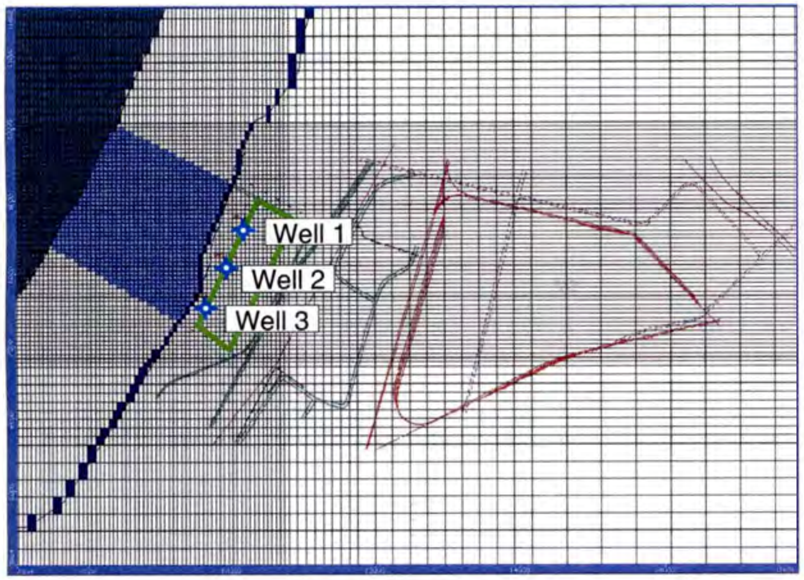
N

GROUNDWATER SERVICES, INC.

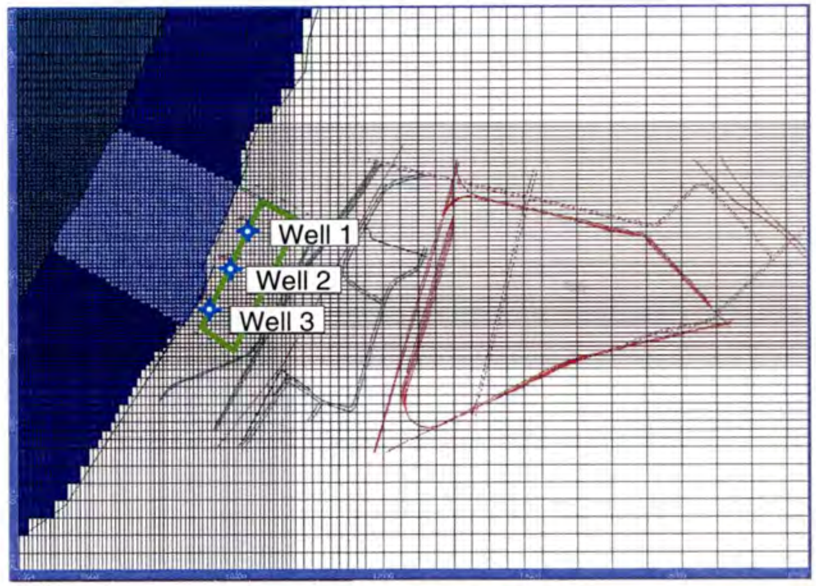
SIMULATED POTENTIOMETRIC SURFACE MAPS DISCHARGE CONTROL STUDY

Site R, W.G. Krummrich Plant, Sauget, Illinois
Solutia Inc.

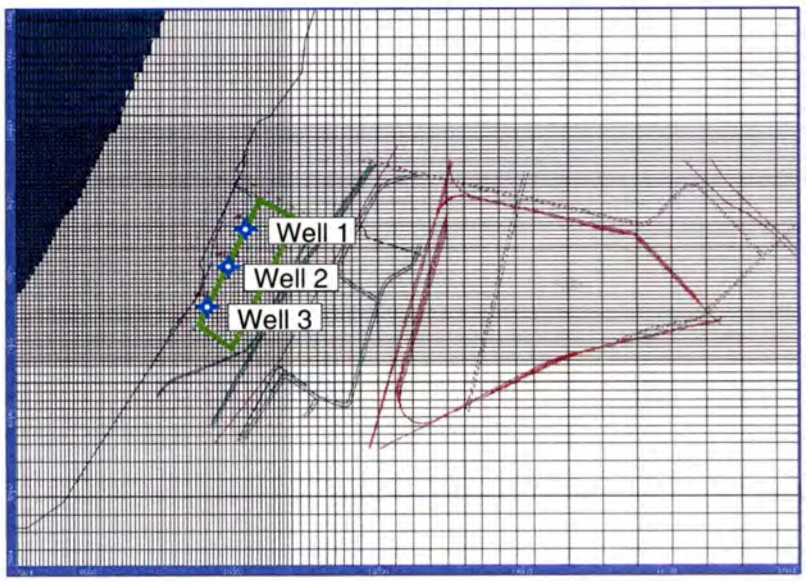
GSI Job No:	G-2561	Drawn By:	CRW
Issued:	11/30/01	Chk'd By:	SKF
Revised:	———	Appv'd By:	CJN
Scale:	As Shown		**FIGURE 4**



Layer 1: Shallow Horizon



Layer 2: Middle Horizon



Layer 3: Deep Horizon

LEGEND

- Pumping well
- River cells in MODFLOW model
- River discharge zone
- Inactive cells
- Approximate area of Site R

SCALE (ft)

0 2000 4000

PRELIMINARY

GROUNDWATER SERVICES, INC.

LOCATION OF RIVER DISCHARGE ZONE AND DISCHARGE CONTROL WELLS

DISCHARGE CONTROL STUDY

Site R, W.G. Krummrich Plant, Sauget, Illinois
Solutia Inc.

GSI Job No:	G-2561	Drawn By:	CRW
Issued:	11/30/01	Chk'd By:	SKF
Revised:	12/21/02	Appv'd By:	CJN
Scale:	As Shown		**FIGURE 5**

N

LEGEND

- Discharge control well
- River cells in MODFLOW model
- Approximate area of Site R
- Head equipotential contour (ft)

PRELIMINARY

NOTES:
1) Grid size in Site R = 60 ft x 60 ft
2) River elevation is 291 ft MSL

SCALE (ft)

0 200 400

GROUNDWATER SERVICES, INC.

EVALUATION OF DOWNGRADIENT CAPTURE ZONE DISCHARGE CONTROL STUDY

Site R, W.G. Krummrich Plant, Sauget, Illinois
Solutia Inc.

GSI Job No:	G-2561	Drawn By:	CRW
Issued:	11/30/01	Chk'd By:	SKF
Revised:	12/21/02	Appv'd By:	CJN
Scale:	As Shown	**FIGURE 6**	

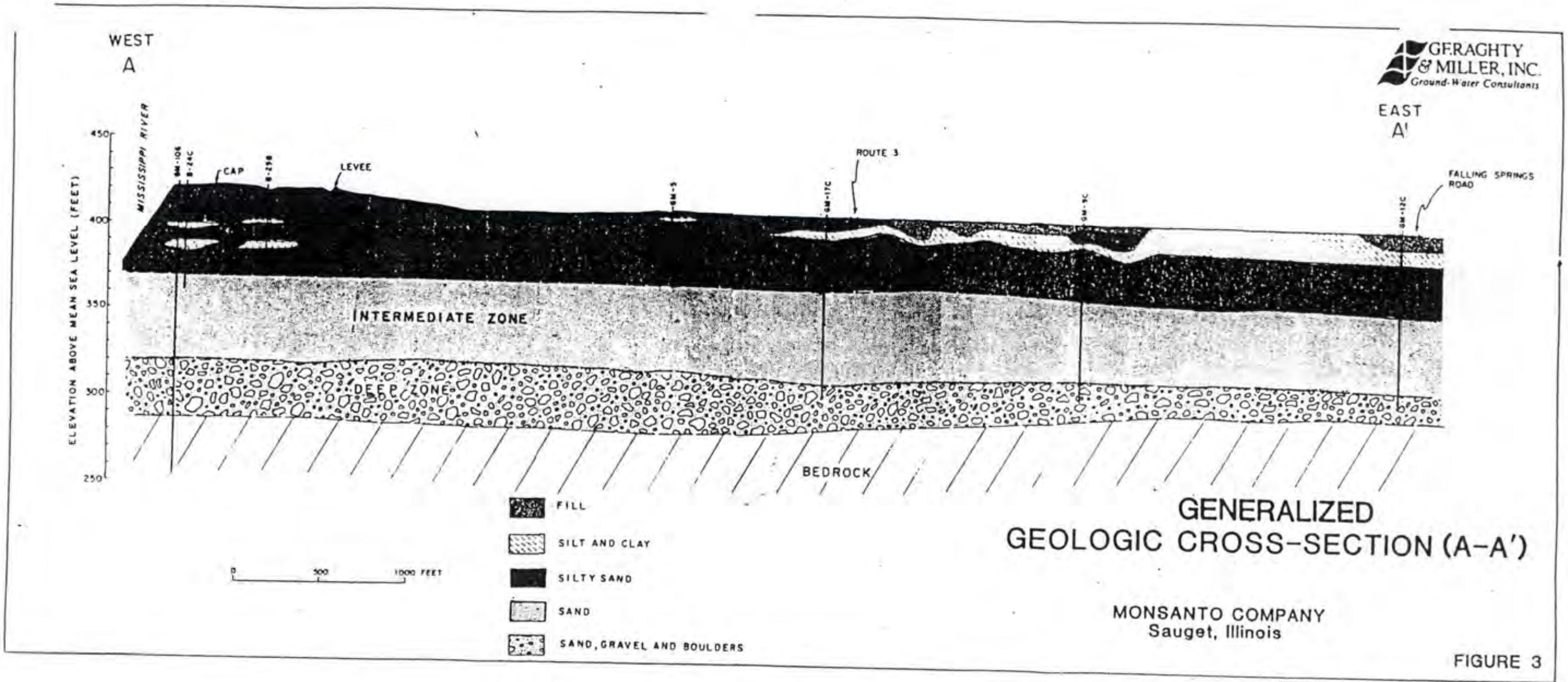
GSI Job No. G-2561
January 24, 2002

GROUNDWATER SERVICES, INC.

Discharge Control Study
Site R - W. G. Krummrich Plant
Sauget, Illinois

Attachments

Attachment A. 1	Geologic Cross-Section, Geraghty & Miller
Attachment A.2.1	Geologic Cross-Section, URS
Attachment A.2.2	Geologic Cross-Section, URS
Attachment A.2.3	Geologic Cross-Section, URS
Attachment A.3'	Geologic Cross-Section, Bergstrom & Walker, 1956
Attachment B	Bedrock Elevation
Attachment C	US Corps of Engineers, River Map
Attachment D	Average Monthly River Stage

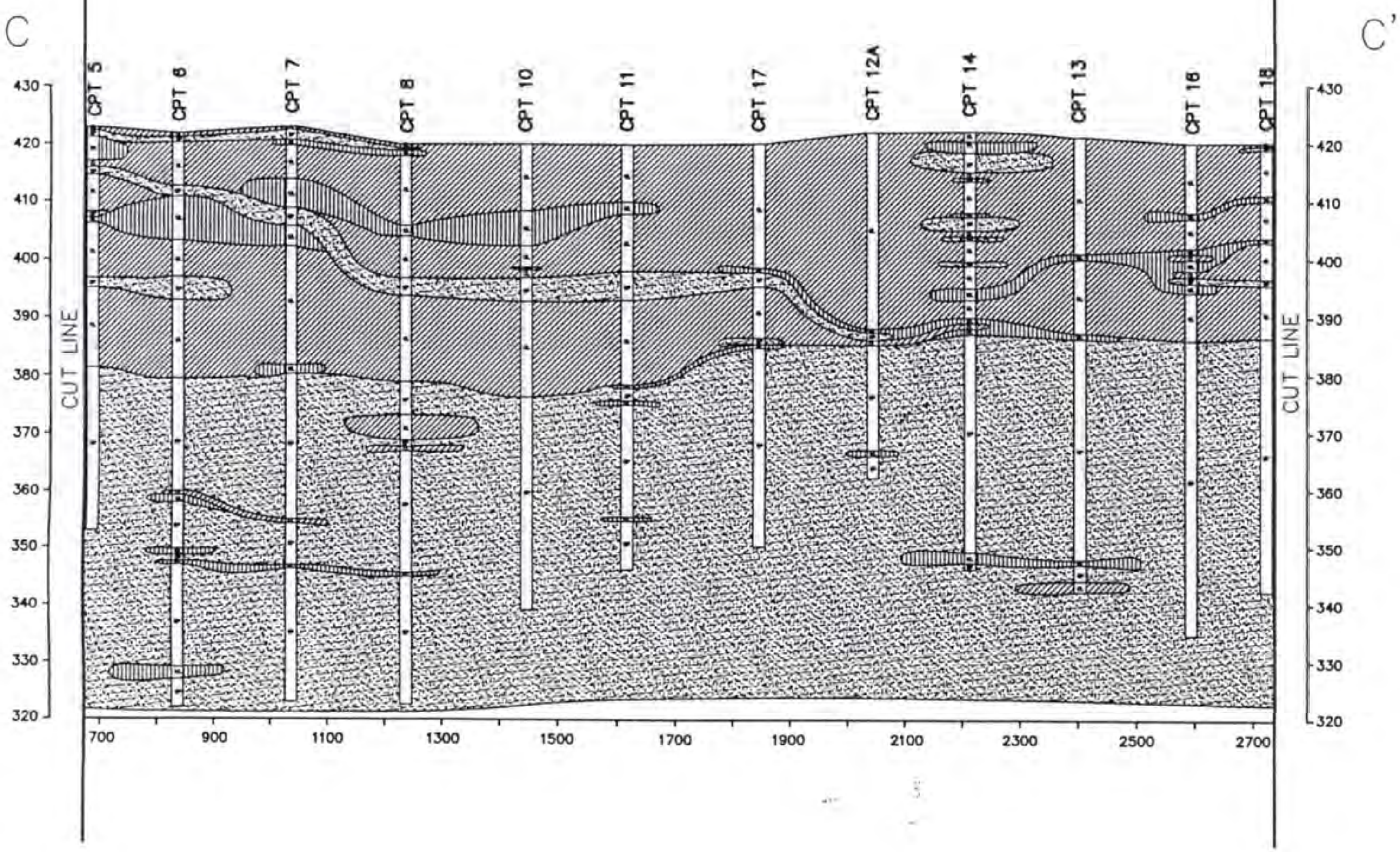


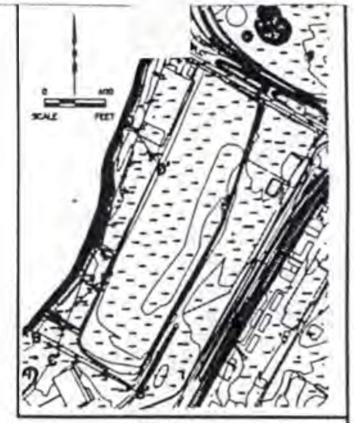
GENERALIZED
GEOLOGIC CROSS-SECTION (A-A')

MONSANTO COMPANY
Sauget, Illinois

FIGURE 3

Attachment A.1
Geologic Cross-Section
Geraghty & Miller





KEY MAP

LEGEND

CLAY

SILT

SAND

CPT-1 CONE PENETROMETER TEST NUMBER

VERTICAL SCALE 1"=10'
HORIZONTAL SCALE 1"=100'

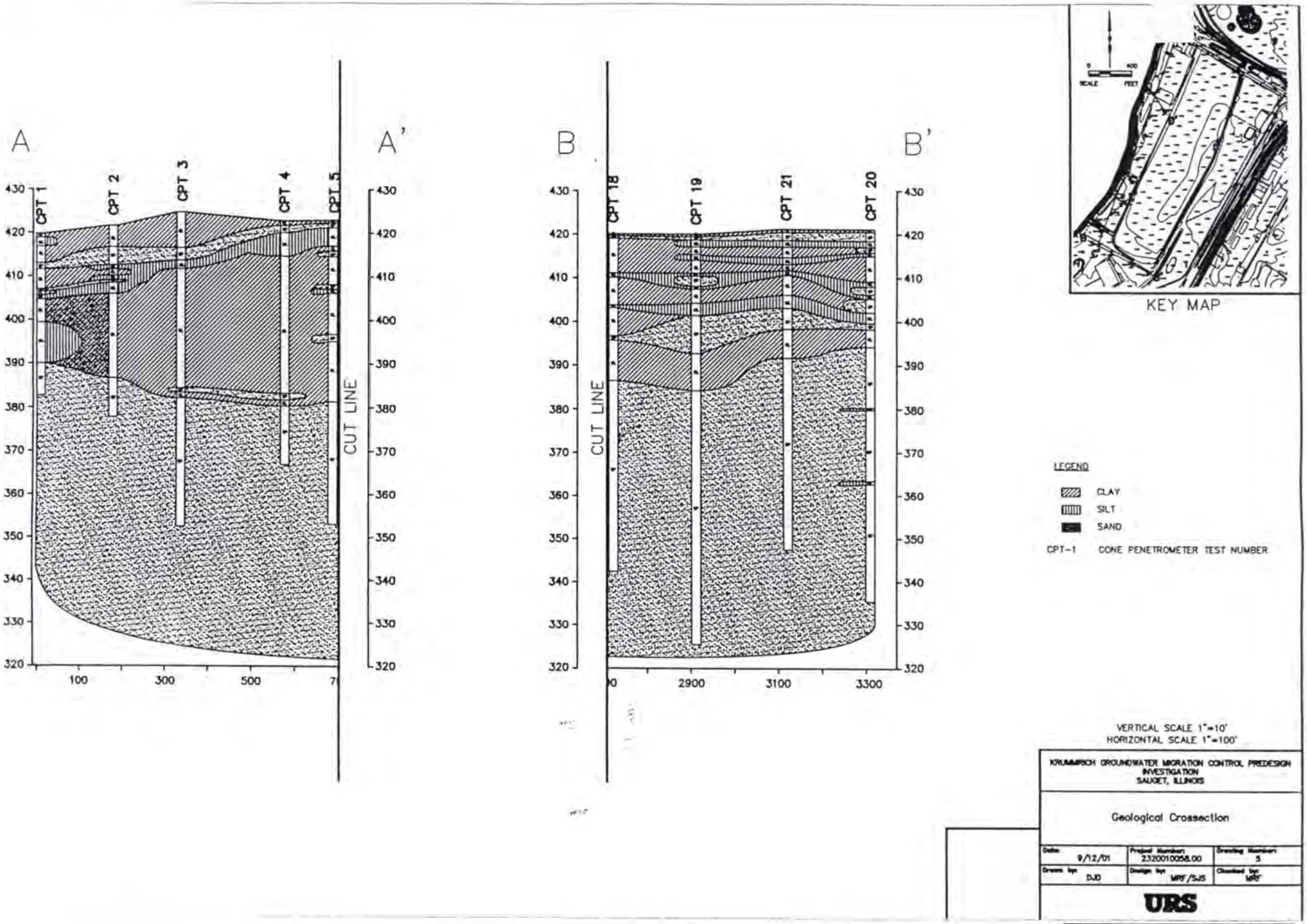
KRUMMRICH GROUNDWATER MIGRATION CONTROL PREDESIGN INVESTIGATION
SAUGET, ILLINOIS

Geological Crossection

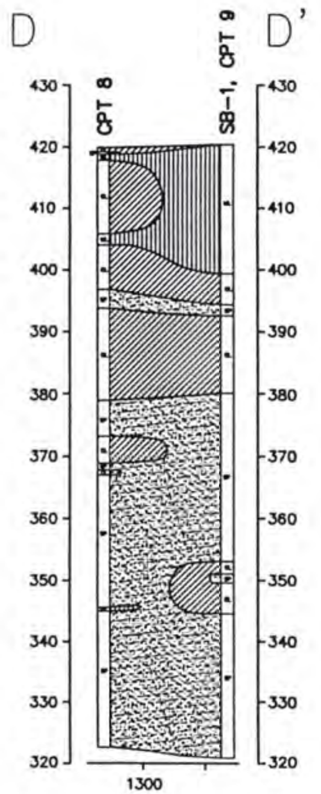
Date: 9/12/01	Project Number: 2320010058.00	Drawing Number: 6
Drawn by: DJD	Design by: MRF/SJS	Checked by: MRF

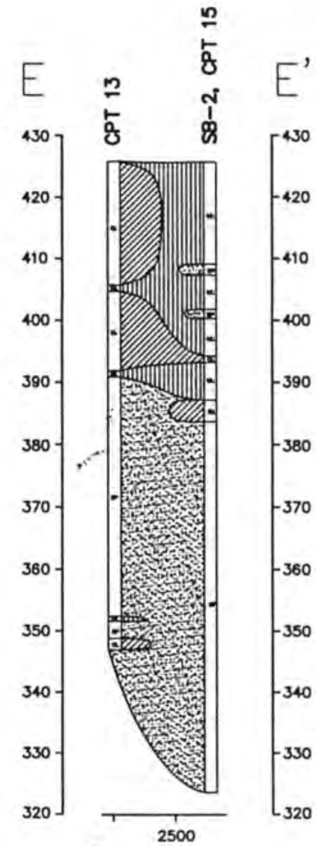
URS

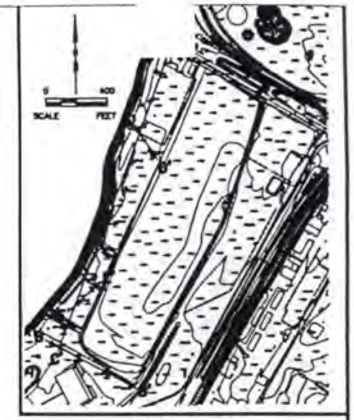
Attachment A.2.1
Geologic Cross-Section
URS



Attachment A.2.2
Geologic Cross-Section
URS







KEY MAP

LEGEND

- CLAY
- SILT
- SAND

CPT-1 CONE PENETROMETER TEST NUMBER

VERTICAL SCALE 1"=10'
HORIZONTAL SCALE 1"=100'

KRUMMRICH GROUNDWATER MIGRATION CONTROL PREDESIGN INVESTIGATION SAUGET, ILLINOIS		
Geological Crossection		
Date: 9/12/01	Project Number: 2320010058.00	Drawing Number: 7
Drawn by: DJD	Design by: MRF/SJS	Checked by: MRF
URS		

Attachment A.2.3
Geologic Cross-Section
URS

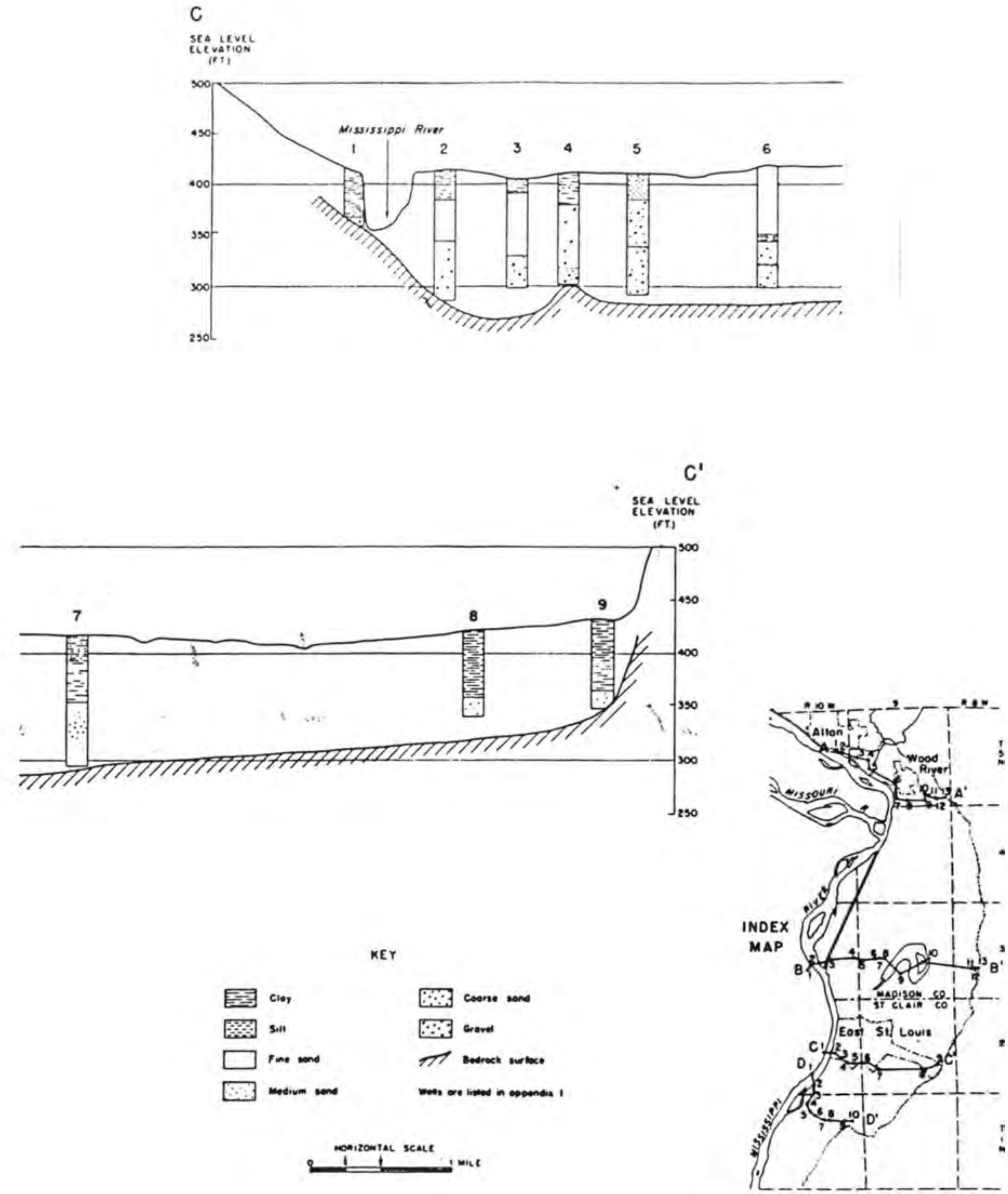


FIG. 4 —(cont.)

Attachment A.3′
Geologic Cross-Section
Bergstrom and Walker, 1956

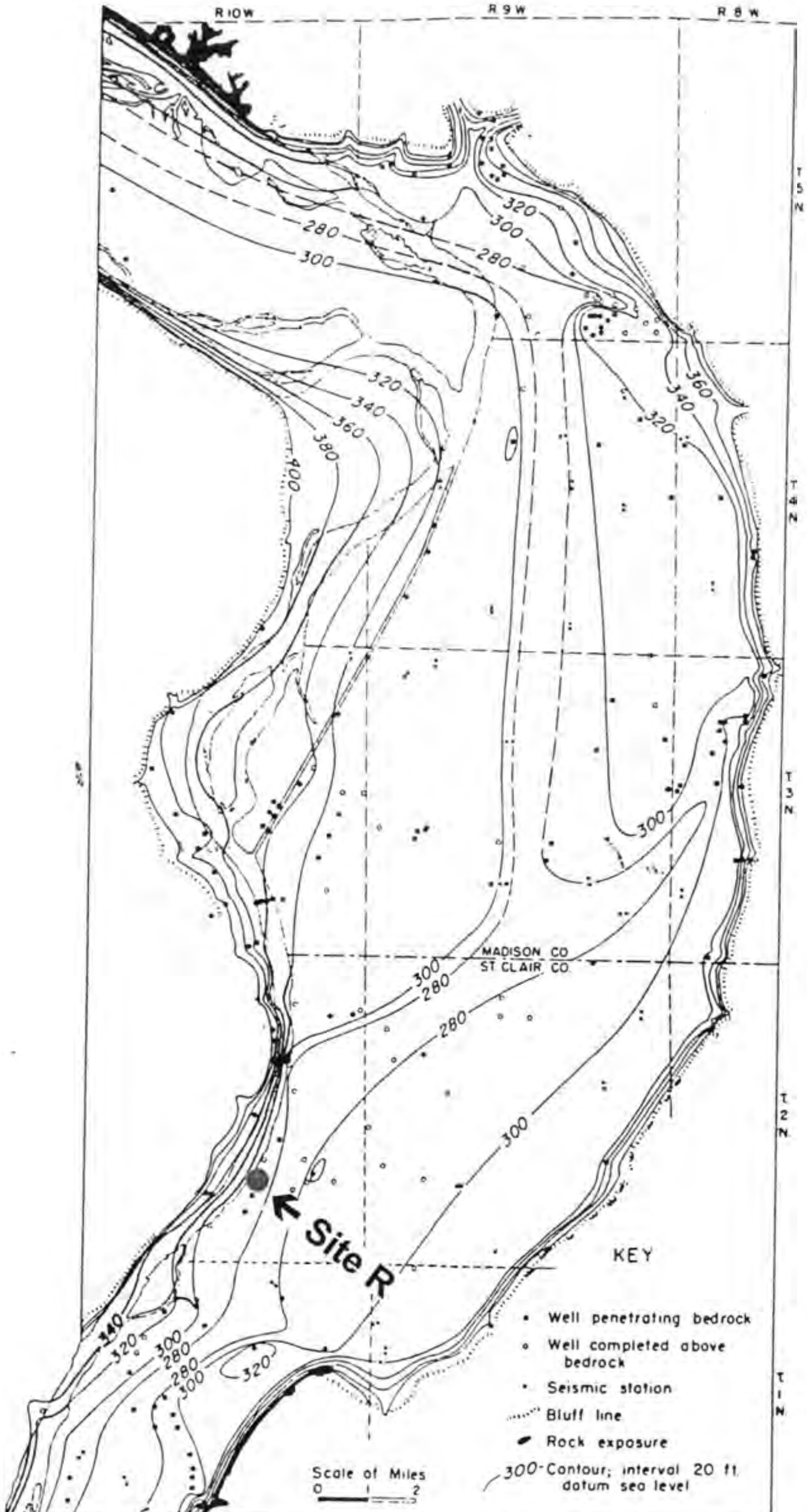
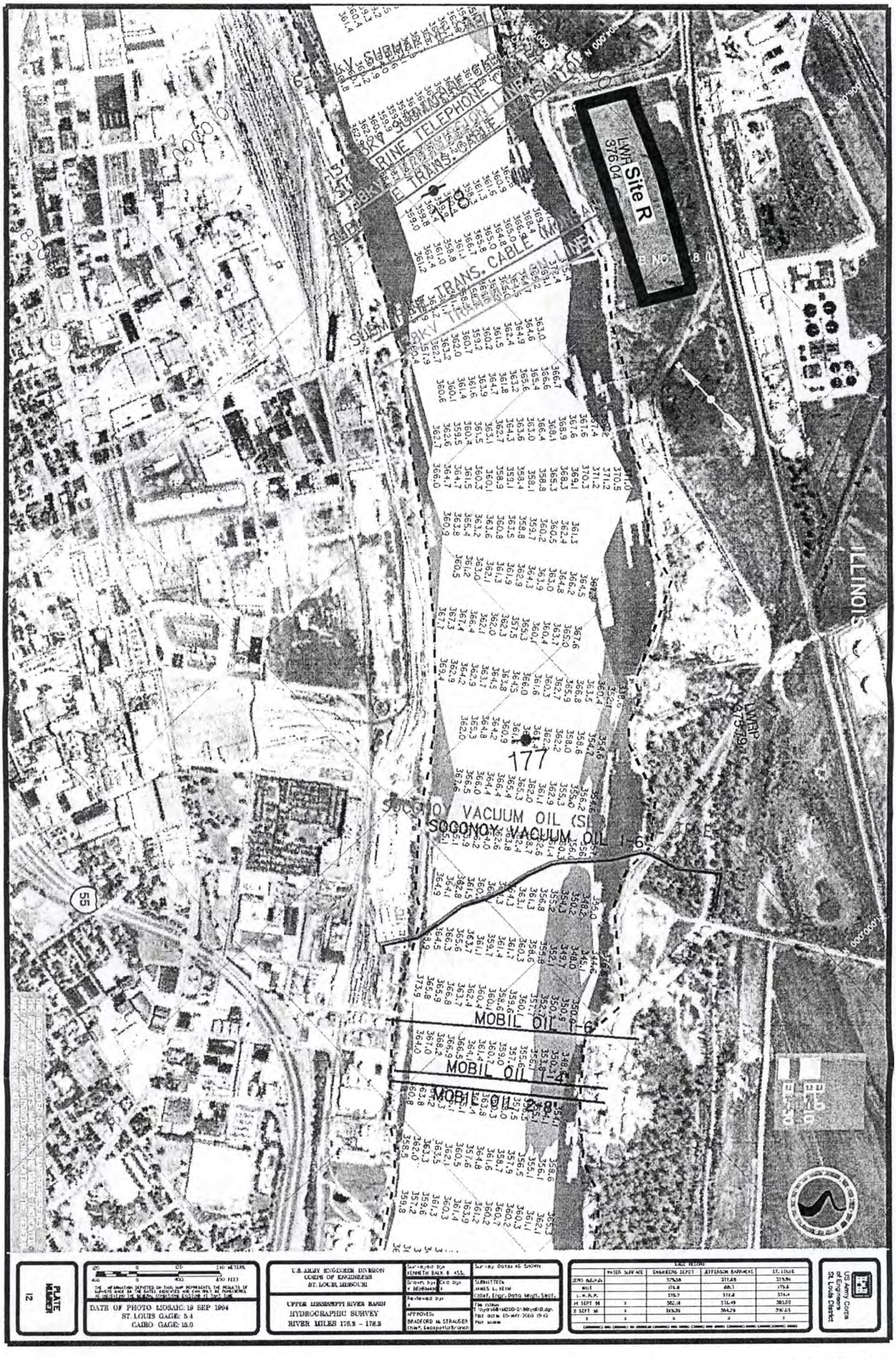
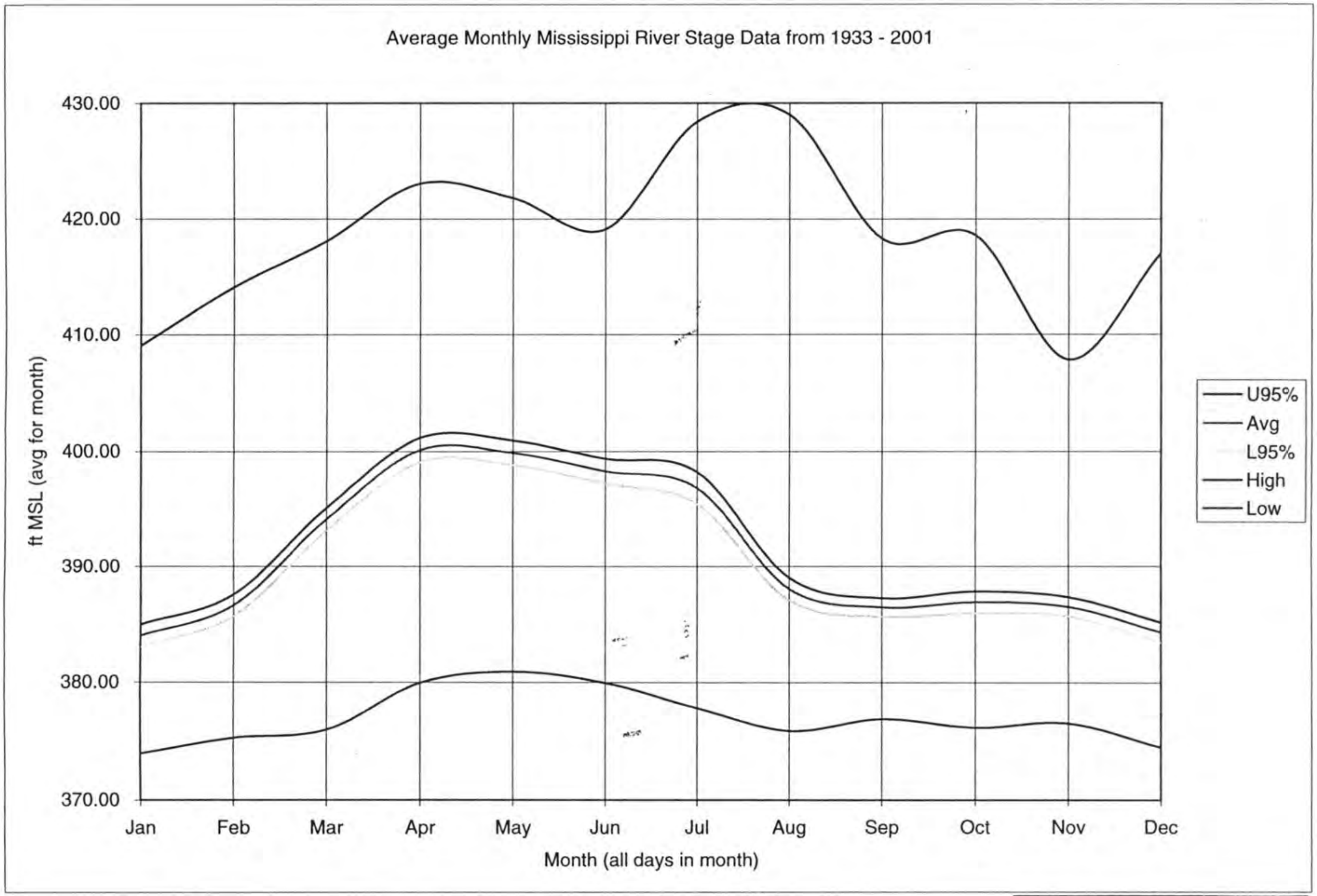


FIG. 2.—Bedrock surface map of the East St. Louis area, Ill.

Attachment B
Bedrock Elevation



Attachment C'
US Corps of Army Engineers - River Map



Appendix D
Average Monthly River Stage

REPORT OF INVESTIGATION 51

Ground-Water Development in East St. Louis Area, Illinois

by R. J. SCHICHT

Printed by authority of the State of Illinois—CH. 127, IRS, Par. 58.29

STATE OF ILLINOIS

Hon. Otto Kerner, Governor

DEPARTMENT OF REGISTRATION AND EDUCATION

John C. Watson, Director

STATE WATER SURVEY DIVISION

William C. Ackermann, Chief

URBANA

1965

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ILLUSTRATIONS

FIGURE		PAGE

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Ground-Water Development in East St. Louis Area, Illinois

by R. J. Schicht

ABSTRACT

The East St. Louis area extends along the valley lowlands of the Mississippi River in southwestern Illinois and covers about 175 square miles. Large supplies of ground water chiefly for industrial development are withdrawn from permeable sand and gravel in unconsolidated valley fill in the area. The valley fill composed of recent alluvium and glacial valley-train material has an average thickness of 120 feet. The coefficient of permeability of the valley fill commonly exceeds 2000 gallons per day per square foot (gpd/sq ft); the coefficient of transmissibility ranges from 50,000 to 300,000 gallons per day per foot (gpd/ft). The long-term coefficient of storage of the valley fill is in the water-table range.

Pumpage from wells increased from 2.1 million gallons per day (mgd) in 1900 to 110.0 mgd in 1956 and was 105.0 mgd in 1962. Of the 1962 total pumpage, 91.1 percent was industrial; 6.4 percent was for public water supplies; 2.3 percent was for domestic uses; and 0.2 percent was for irrigation. Pumpage is concentrated in five major pumping centers: the Alton, Wood River, Granite City, National City, and Monsanto areas.

As the result of heavy pumping, water levels declined about 50 feet in the Monsanto area, 40 feet in the Wood River area, 20 feet in the Alton area, 15 feet in the National City area, and 10 feet in the Granite City area from 1900 to 1962. From 1957 to 1961 water levels in the Granite City area recovered about 50 feet where pumpage decreased from 31.6 to 8.0 mgd. Pumping of wells and draining of lowlands have considerably reduced ground-water discharge to the Mississippi River, but have not reversed at all places the natural slope of the water table toward that stream. In the vicinity of some pumping centers, the water table has been lowered below the river and other streams, and induced infiltration of surface water is occurring.

Recharge directly from precipitation based on flow-net analysis of piezometric maps varies from 299,000 to 475,000 gallons per day per square mile (gpd/sq mi). Subsurface flow of water from bluffs bordering the area into the aquifer averages about 329,000 gallons per day per mile (gpd/mi) of bluff. Infiltration rates of the Mississippi River bed according to the results of aquifer tests range from 344,000 to 37,500 gallons per day per acre per foot (gpd/acre/ft). Approximately 50 percent of the total pumpage in 1962 was derived from induced infiltration of surface water.

An electric analog computer consisting of an analog model and excitation-response apparatus was constructed for the East St. Louis area so that the consequences of further development of the aquifer could be forecast. The accuracy and reliability of the analog computer were established by comparing actual water-level data with piezometric surface maps prepared with the analog computer.

The analog computer was used to estimate the practical sustained yields of existing pumping centers. Assuming that critical water levels will occur when pumping water levels are below tops of screens and/or more than one-half of the aquifer is dewatered, the practical sustained yields of all existing pumping centers exceed present withdrawals. Pumpage in the Monsanto area probably will exceed the practical sustained yield by 1966; the practical sustained yield of other pumping centers probably will not be reached until after 1980. The analog computer was also used to describe the effects of a selected scheme of development and to determine the potential yield of the aquifer under an assumed pumping condition.

INTRODUCTION

The East St. Louis area has been one of the most favorable ground-water areas in Illinois. It is underlain at depths of 170 feet or less by sand and gravel aquifers that have been prolific sources of water for more than 50 years. The available ground-water resources have promoted industrial expansion of the area and also facilitated urban growth.

The tremendous industrial growth in the East St. Louis area has brought about local problems of water supply. Heavy concentrated pumpage in the Granite City area caused water levels to decline to critical stages during an extended dry period (1952-1956). As a result, an industry was forced to abandon its well field and construct a pipe line to the Mississippi River for its water supply.

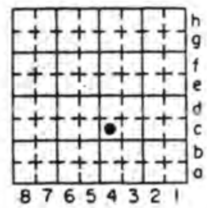
This report presents a quantitative evaluation of the ground-water resources of the East St. Louis area and is based on all data on file at the State Water Survey and in other published reports. The geohydrologic characteristics of the ground-water reservoir are given along with an analysis of past, present, and probable future development of ground-water resources. Basic geologic, hydrologic, and chemical data, maps, and interpretations applicable to local problems and to regional and long-range interpretations are presented to provide a basis for water-resource planning and a guide to the development and conservation of ground water in the area.

Although this report summarizes present-day knowledge of ground-water conditions in the East St. Louis area, it must be considered a preliminary report in the sense that it is part of a continuing study of the East St. Louis ground-water resources. The conclusions and interpretations in this report may be modified and expanded from time to time as more data are obtained.

The State Water Survey accelerated its program of ground-water investigation in the East St. Louis area in 1941 after alarming water-level recessions were observed by local industries especially at Granite City. Water-level data for the period 1941 through 1951 were summarized and the ground-water withdrawals in 1951 were discussed by Bruin and Smith (1953). The ground-water geology of the area has been described by the State Geological Survey (Bergstrom and Walker, 1956). Ground-water levels and pumpage in the area during the period 1890 through 1961 were discussed by Schicht and Jones (1962). Other reports pertaining to the ground-water resources of the East St. Louis area are listed in the references at the end of this report.

Well-Numbering System

The well-numbering system used in this report is based on the location of the well, and uses the township, range, and section for identification. The well number consists of five parts: county abbreviation, township, range, section, and coordinate within the section. Sections are divided into rows of $1/8$-mile squares. Each $1/8$-mile square contains 10 acres and corresponds to a quarter of a quarter of a quarter section. A normal section of 1 square mile contains 8 rows of $1/8$-mile squares; an odd-sized section contains more or fewer rows. Rows are numbered from east to west and lettered from south to north as shown in the diagram.



St. Clair County

T2N, R10W

Section 23

The number of the well shown is: STC 2N10W-23.4c. Where there is more than one well in a 10-acre square they are identified by arabic numbers after the lower case letter in the well number.

There are parts of the East St. Louis area where section lines have not been surveyed. For convenience in locating observation wells, normal section lines were assumed to exist in areas not surveyed.

The abbreviations for counties discussed in this report are:

Madison MAD   Monroe MON   St. Clair STC

In the listing of wells owned by municipalities, the place-name is followed by V, T, or C in parentheses to indicate whether it is a village, town, or city, except where the word City is part of the place-name.

Acknowledgments

This study was made under the general supervision of William C. Ackermann, Chief of the Illinois State Water Survey, and Harman F. Smith, Head of the Engineering Section. William C. Walton, formerly in charge of ground-water research in the Engineering Section, aided in interpretation of hydrologic data and reviewed and criticized the final manuscript. E. G. Jones, field engineer, collected much of the water-level, pumpage, and specific-capacity data, and aided indirectly in preparing this report.

Many former and present members of the State Water Survey assisted in the collection of data, wrote earlier special reports which have been used as reference material, or contributed other indirect assistance to the writer. Grateful acknowledgment is made, therefore, to the following engineers: G. E. Reitz, Jr., R. R. Russell, Sandor

Conallany, W. H. Walker, T. A. Prickett, Jack Bruin, J. P. ·, R. E. Aten, H. G. Rose, and O. E. Michaels. J. W. ..other prepared the illustrations.

This report would have been impossible without the generous cooperation of officials of municipalities and industries, consulting engineers, water well contractors, and irrigation and domestic well owners who provided information on wells, water levels, and pumpage.

GEOGRAPHY

The East St. Louis area, known locally as the "American Bottom," is in southwestern Illinois and includes portions of Madison, St. Clair, and Monroe Counties. It encompasses the major cities of East St. Louis, Granite City, and Wood River, and extends along the valley lowlands of the Mississippi River from Alton south beyond Cahokia as shown in figure 1. The area covers about 175 square miles and is approximately 30 miles long and 11 miles wide at the widest point. Included is an area south of Prairie Du Pont Floodway containing Dupo and East Carondelet.

Topography and Drainage

Most of the East St. Louis area lies in the Till Plains ·tion of the Central Lowland Physiographic Province enneman, 1914; and Leighton, Ekblaw, and Horberg, 1948). The extreme southwestern part of St. Clair County and the western part of Monroe County lie in the Salem Plateau Section.

Much of the area lies in the flood plain of the Mississippi River; the topography consists mostly of nearly level bottomland. Along the river channel the flood plain slopes from an average elevation of 415 feet near Alton to 405 feet near Dupo. In the northern part of the area, terraces stand above the flood plain. A terrace that extends from East Alton to Roxana is at an elevation of 440 to 450 feet or about 25 to 35 feet above the flood plain. North of Horseshoe Lake much of the area is above the flood plain at elevations ranging from 420 to 435 feet.

The elevation of the land surface near the eastern bluff is 30 to 50 feet higher than the general elevation of the valley bottom. The bluff, along the eastern edge of the valley bottom, rises abruptly 150 to 200 feet above the lowland. The topography immediately east of the bluff consists of rather rugged uplands.

Monks Mound, which rises 85 feet above the flood plain, is the largest of a group of mounds just east of Fairmont City. The shape of the mounds indicates an ·rtificial origin; however, some of them may be remnants f an earlier higher flood plain (Bergstrom and Walker, 1956).

Drainage is normally toward the Mississippi River and its tributaries; Wood River, Cahokia Diversion Channel, Cahokia Canal, and Prairie Du Pont Floodway. The tributaries drain much of the flood plain and the uplands bordering the flood plain. The valley bottom is protected from flooding by a system of levees that fronts the Mississippi River and the Chain of Rocks Canal and flanks the main tributaries. However, flooding does occur in parts of the area because drainage facilities which convey and store major flood runoff from the flood plain and the upland watersheds are inadequate (Illinois Division of Waterways, 1950). The southeastern part of the area near Cahokia, Centreville, and Grand Marais State Park is particularly affected by flooding. Figure 1 shows areas flooded after heavy rainfall on May 5, 6, 7, 8, and 19, 1961.

Prior to settlement of the East St. Louis area, floodwaters from the Mississippi River and its tributary streams, Wood River, Cahokia Creek, Canteen Creek, Schoenberger Creek, and Prairie Du Pont Creek, frequently inundated large sections of the valley bottom. The water table was near the surface and poorly drained areas were widespread. Development of the area led to a system of drainage ditches, levees, canals, and channels. According to Bruin and Smith (1953) the natural lake area between 1907 and 1950 was reduced by more than 40 percent and 40 miles of improved drainage ditches were constructed during the same period; this had an effect of lowering ground-water levels by an estimated 2 to 12 feet.

The present drainage system is shown in figure 2. Much of the flow from the upland areas east of the bluff is diverted into four channels that traverse or flank the valley bottom, thence flow to the Mississippi River. The four channels are Wood River, Cahokia Diversion Channel, Prairie Du Pont Floodway, and Canal No. 1.

Wood River carries flow from the confluence of the East and West Forks of Wood River north of East Alton south-southwest to the Mississippi River. Much of the channel of Wood River is leveed.

The Cahokia Diversion Channel intercepts flow from Cahokia and Indian Creeks in sec 7, T4N, R8W, Madison County, and diverts it westward to the Mississippi River.

Prairie Du Pont Floodway is a relocated and improved channel of Prairie Du Pont Creek and conveys runoff from Canal No. 1 and Prairie Du Pont Creek near Stolle westward to the Mississippi River. In addition it carries flow from the valley bottom drainage area north of Prairie Du Pont Creek and from Harding Ditch.

Canal No. 1 intercepts flow from several small upland

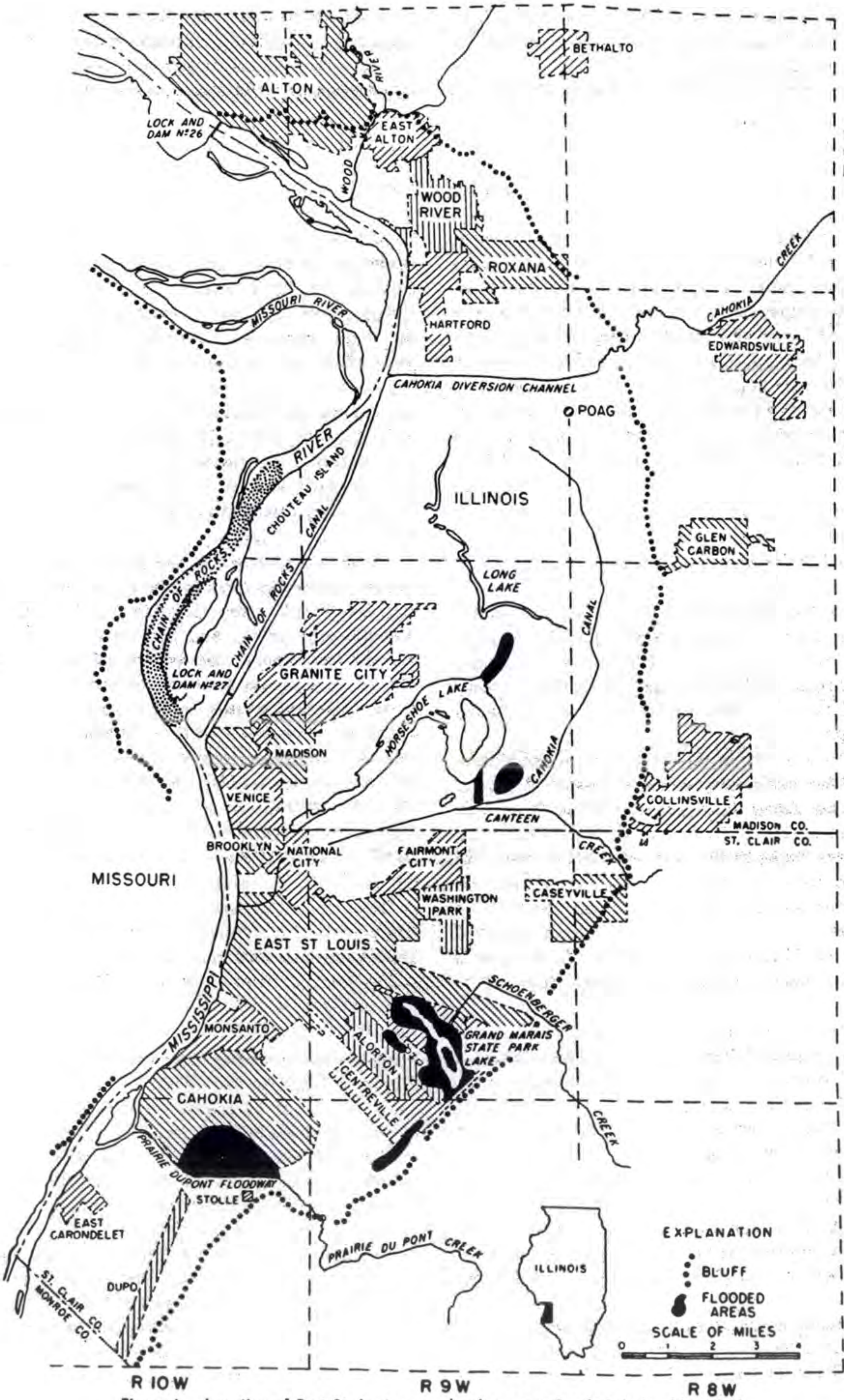


Figure 1. Location of East St. Louis area showing areas flooded during May 1961

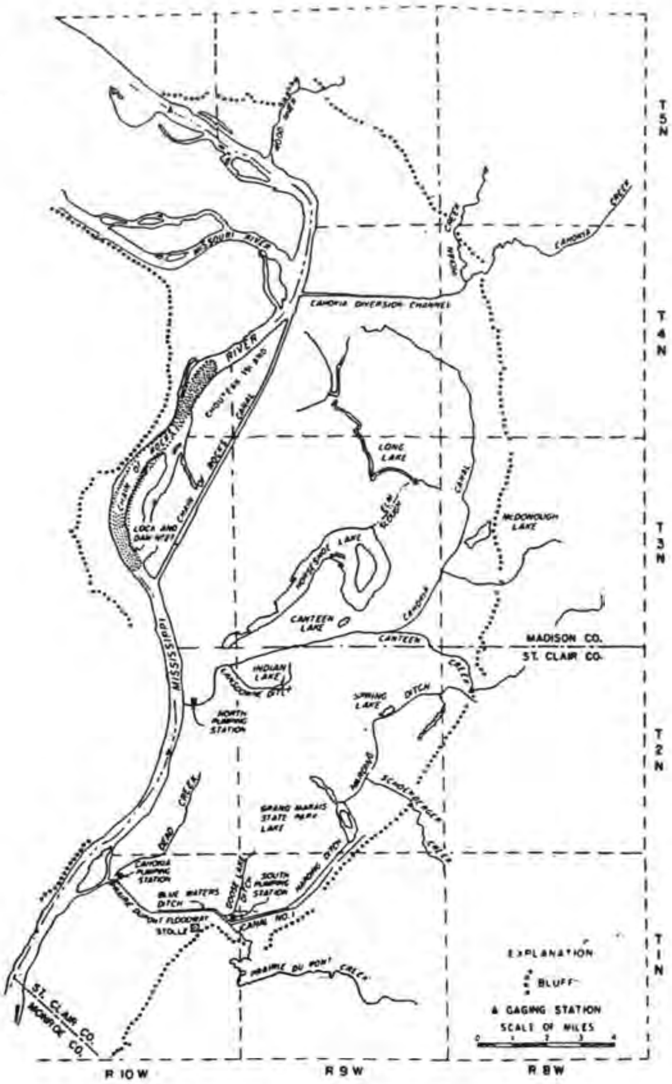


Figure 2. Drainage system and locations of stream-gaging stations

streams between Prairie Du Pont Floodway and the southern edge of Centreville and discharges the flow into the floodway.

The valley bottom is drained through Indian Creek, several small ditches north of the Cahokia Diversion Channel, Long Lake, Cahokia Canal, Lansdowne Ditch, Harding Ditch, the Blue Waters-Goose Lake Ditch system, and the Dead Creek-Cahokia drainage system. In addition, closed storm sewer systems drain much of the urban areas within the valley bottom.

Long Lake drains much of the area to the north of Horseshoe Lake. During periods of overflow it drains into Horseshoe Lake through Elm Slough.

The Cahokia Canal consists of an improved and leveed channel along the old course of Cahokia Creek. The canal begins in sec 14, T4N, R9W, flows southeasterly to sec 31, T4N, R8W, and then southwesterly around the southern end of Horseshoe Lake, through National City and the northwestern corner of East St. Louis to the Mississippi River. Discharge to the Mississippi River is by gravity flow during periods when the stage of the Mississippi River is low; when the river is at flood stage, water is pumped from Cahokia Canal to the river at the North Pumping Station. Runoff in excess of the storage capacity of Cahokia Canal or of the pumping station is stored temporarily in Indian and Horseshoe Lakes until it can be discharged into the river. The principal tributaries to the canal are Long Lake (by way of Horseshoe Lake), Lansdowne Ditch, Canteen Creek, and several small streams to the east.

Harding Ditch begins at Caseyville and flows southwesterly to Park Lake in Grand Marais State Park, which acts as a regulating reservoir, thence to Prairie Du Pont Floodway. Discharge to the Mississippi River is either by gravity flow or pumps at the South Pumping Station.

The Dead Creek-Cahokia drainage system drains most of the Monsanto and Cahokia areas. The outlet of the system is to the Prairie Du Pont Floodway at the Cahokia Pumping Station.

The Blue Waters-Goose Lake Ditch system drains the area east of Cahokia, southwest of Centreville, and northwest of Harding Ditch and Prairie Du Pont Floodway. Goose Lake Ditch discharges into Blue Waters Ditch near Harding Ditch. Blue Waters Ditch can discharge into Prairie Du Pont Floodway or Harding Ditch when the floodway is at low stage; when the stage of the floodway is high, runoff is stored temporarily in Blue Waters Ditch and adjacent low areas.

Numerous lakes were formed in the flood plain by the meandering of the Mississippi River. Many of the lakes have been drained and the original lake bottoms are now being cultivated. Table 1 gives data on the more important lakes now in existence.

Table 1. Areas and Water-Surface Elevations of Lakes*

Lake	Approximate surface area when full (acres)	Approximate water surface elevation when full (ft above msl)
McDonough	75	404
Long	85	415
Horseshoe	2500	402
Canteen	105	403
Park	990	405.5
Spring	10	410

*From *Illinois Division of Waterways (1950)*

The average gradient of the Mississippi River from Alton to Dupo is about 6 inches per mile. The average gradients of Wood River, Cahokia Diversion Channel, Cahokia Canal, and Prairie Du Pont Floodway are given in table 2. The gradients of streams draining the uplands east of the bluff are much greater, ranging from about 6 feet per mile for Cahokia Creek to about 30 feet per mile for Schoenberger Creek.

The Chain of Rocks Canal was constructed to bypass the reach of the Mississippi River known as Chain of

Table 2. Average Gradients of Tributaries to Mississippi River

Tributary	Gradient (ft per mi)
Wood River	5
Cahokia Diversion Channel	2
Cahokia Canal	1.7
Prairie Du Pont Floodway	1.6

Rocks Reach (figure 1), which was difficult to navigate because the velocity of the river sometimes exceeded 12 feet per second. In addition, the navigable depth in Chain of Rocks Reach was reduced to 5.5 feet when the stage of the river was low. The canal, which was opened to river traffic on February 7, 1953, is 300 feet wide at the bottom and about 550 feet wide at the top, and has a total length of 8.4 miles. In the vicinity of Granite City the canal was widened, for a distance of 6750 feet, to a bottom width of 700 feet. A depth of slightly less than 15 feet at minimum low water stage is provided at the lower end of the canal downstream from Lock No. 27. At the upstream entrance of the canal, a minimum depth of 10.4 feet is provided.

The locations of stream gages in the East St. Louis area are shown in figure 2. The U. S. Geological Survey measures the discharge of the Mississippi River at Alton, and at St. Louis. The discharges of Indian Creek near Wanda and Canteen Creek near Caseyville are also measured by the U.S. Geological Survey, and the discharge of Long Lake near Stallings was measured from December 1938 to December 1949. Extremes and average discharges of streams are given in table 3.

During the 1952 to 1956 drought the average discharge of Indian and Canteen Creeks was reduced considerably. The average daily discharge was 6.23 cubic feet per second (cfs) in Indian Creek at Wanda and 5.81 cfs in Canteen Creek near Caseyville. There was no flow in these streams during many days in the summer and fall months of the drought period.

The flow of the Mississippi River in the East St. Louis area is affected by many reservoirs and navigation dams in the upper Mississippi River Basin and by many reservoirs and diversions for irrigation in the Missouri River Basin. Along the reach of the Mississippi River from Alton to Dupo the flow of the river is affected by Lock and Dam No. 26 at Alton, the Chain of Rocks Canal, and Lock and Dam No. 27 at Granite City on the canal. There is a low water dam on the Mississippi River south of the northern end of Chain of Rocks Canal.

Floodwaters from the Missouri River enter the Mississippi River above the gaging station at Alton when levees along the Missouri River are overtopped. Overflow from the Missouri River was estimated by the U. S. Geological Survey and is given in table 4.

Mississippi River stages in the East St. Louis area are measured daily at Lock and Dam No. 26 at Alton; at Hartford, Illinois; Chain of Rocks, Missouri; Lock No. 27 at Granite City, Illinois; Bissell Point, Missouri; St. Louis, Missouri; and the Engineer Depot, Missouri. The elevation of the maximum river stage at Alton was estimated to be 432.10 feet and occurred in June 1844; the elevation of the minimum stage was 390.50 feet on January 27, 1954. The elevation of the maximum river stage

Table 3. Streamflow Records

Stream	Drainage area (sq mi)	Location of gaging station	Maximum discharge (cfs) and date of occurrence	Minimum discharge (cfs) and date of occurrence	Average discharge (cfs) and length of record	Average discharge (cfs) during 1952-1956 drought
Mississippi River	171,500	At Alton, mile 202.7 upstream from Ohio River	437,000 May 24, 1943	7,960 November 7, 1948	93,130 33 years	
Mississippi River	701,000	At St. Louis mile 180.0 upstream from Ohio River	1,300,000* June 1844	18,000 December 21-23, 1863	174,700 99 years	
Indian Creek	37	At Wanda, SE ¼ NW ¼ sec 31, T5N, R8W	9,340 August 15, 1946	0†	24.8 21 years	6.23
Long Lake	5	At Stallings, NW ¼ NW ¼ sec, 12, T3N, R9W	121 August 18, 1946	0†	2.31 12 years	
Canteen Creek	23	At Caseyville, N ½ NW ¼ sec 8, T2N, R8W	10,200 June 15, 1957	0†	17.5 22 years	5.81

*Estimated
†Zero flow occurred during several periods in drought years

St. Louis was 421.26 feet and occurred on June 27, 4; the elevation of the minimum stage was 373.33 feet on January 16, 1940.

Table 4. Overflow from Missouri River

Period	Overflow for period (ac-ft)	Maximum Date of occurrence	Maximum Overflow (cfs)
May 21-June 4, 1943	1,075,000	May 24, 1943	90,000
April 29-May 13, 1944	891,000	April 30, 1944	90,000
June 29-July 19, 1947	687,000	July 2, 1947	65,000
July 5-31, 1951	2,534,000	July 20, 1951	110,000

Climate

The East St. Louis area lies in the north temperate zone. Its climate is characterized by warm summers and moderately cold winters.

According to the Atlas of Illinois Resources, Section 1 (1958), the average annual precipitation in the East St. Louis area is about 38 inches. Precipitation has been measured at St. Louis since 1837. Graphs of annual and mean monthly precipitation collected by the U. S. Weather Bureau at Lambert Field near St. Louis (1905 to 1962) and at Edwardsville (1930 to 1962) are given in figures 3 and 4, respectively. According to the records at Edwardsville, the months of greatest precipitation (exceeding 3.5 inches) are March through August; December is the month of least precipitation having 2.07 inches.

In addition to precipitation records available for Edwardsville, St. Louis, and Lambert Field, records for different periods are available for the gaging stations given in table 5 within and near the East St. Louis area.

The annual maximum precipitation amounts occurring on an average of once in 5 and once in 50 years are 45 and 57 inches, respectively; annual minimum amounts expected for the same intervals are 31 and 25 inches, respectively. Amounts are based on data given in the Atlas of Illinois Resources, Section 1 (1958).

The mean annual snowfall is about 17 inches. On the average, about 16 days a year have 1 inch or more, and

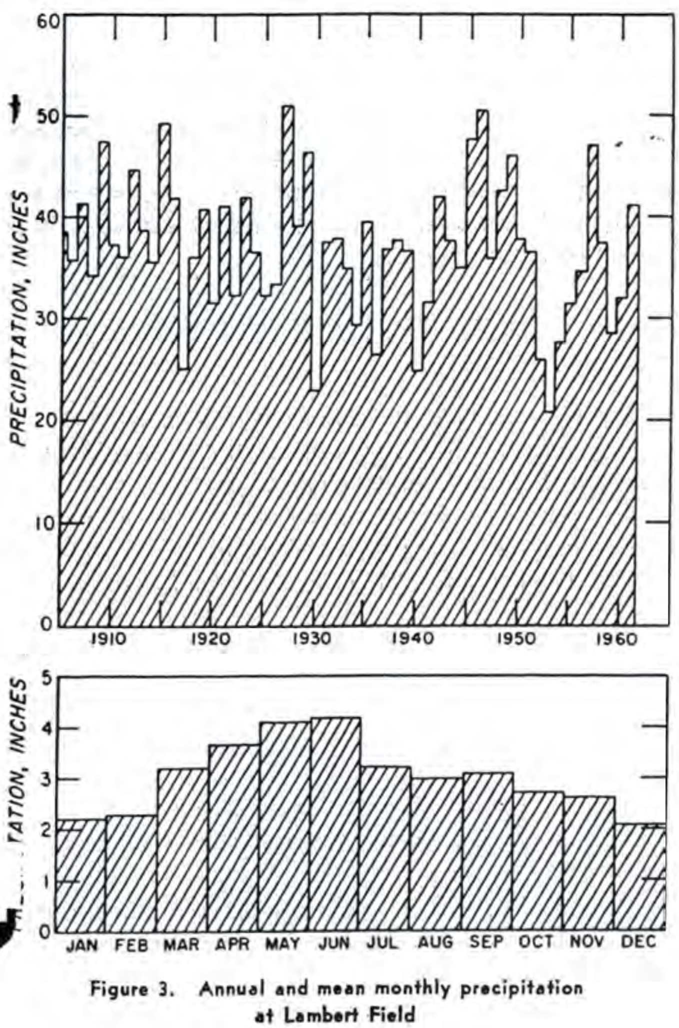


Figure 3. Annual and mean monthly precipitation at Lambert Field

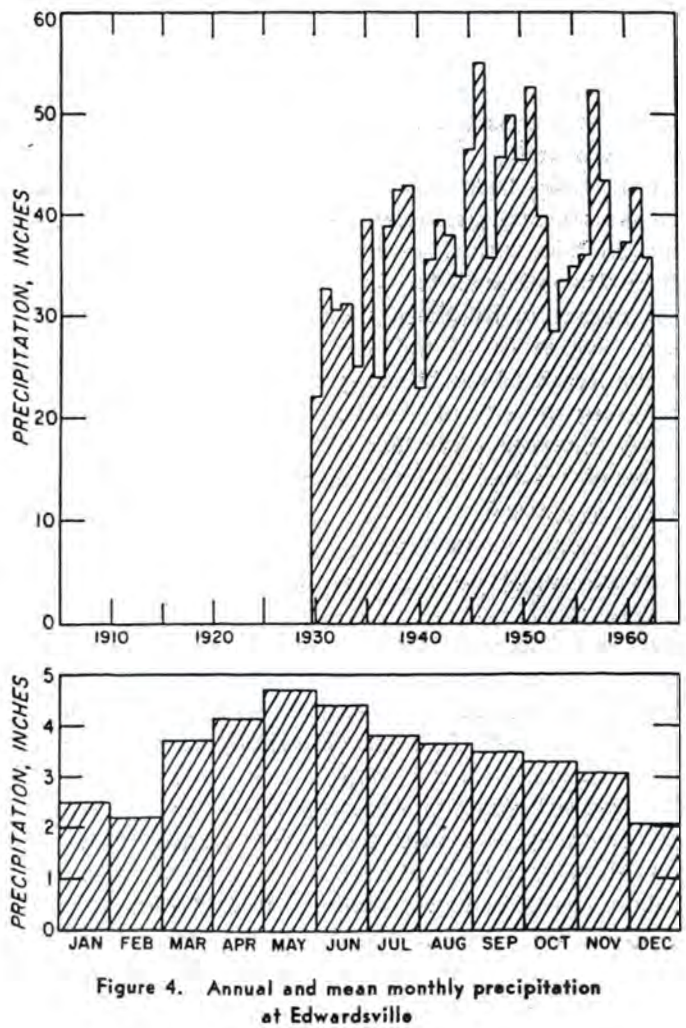


Figure 4. Annual and mean monthly precipitation at Edwardsville

about 8 days a year have 3 inches or more, of ground snow cover.

Based on records collected at Lambert Field, the mean annual temperature is 56.4 F. June, July, and August are the hottest months with mean temperatures of 75.2, 79.6, and 77.8 F, respectively. January is the coldest month with a mean temperature of 32.1 F. The mean length of the growing season is 198 days.

A large part of central and southern Illinois, including the East St. Louis area, experienced a severe drought beginning in the latter part of 1952 (Hudson and Roberts, 1955). For the period 1953 through 1956, cumulative deficiency of precipitation at Edwardsville and Lambert Field was about 22 and 34 inches, respectively.

An intense rainstorm, exceeding 16 inches in 12 hours at places, occurred June 14 and 15, 1957. The storm is discussed in detail by Huff et al. (1958). A Heavy rainstorm also occurred August 14-15, 1946, when over 11 inches were recorded at East St. Louis.

Table 5. Precipitation Gaging Stations

Owner	*Location of gage*
Shell Oil Co.	Wood River
East St. Louis and Interurban Water Co.	Chouteau Island
East Side Levee and Sanitary Dist.	Centreville
East Side Levee and Sanitary Dist.	Collinsville
East Side Levee and Sanitary Dist.	Edgemont
East Side Levee and Sanitary Dist.	Millstadt
Standard Oil Co.	Wood River
Illinois State Water Survey	Lakeside Airport
American Smelting and Refining Co.	Alton
Olin Mathieson Chemical Co.	East Alton
U. S. Weather Bureau	Collinsville
U. S. Weather Bureau	Belleville, Scott Air Force Base
U. S. Weather Bureau	Alton Dam 26
U. S. Weather Bureau	East St. Louis, Parks College

GEOLOGY AND HYDROLOGY

Large supplies of ground water chiefly for industrial development are withdrawn from permeable sand and gravel in unconsolidated valley fill in the East St. Louis area. The valley fill is composed of recent alluvium and glacial valley-train material and is underlain by Mississippian and Pennsylvanian rocks consisting of limestone and dolomite with subordinate amounts of sandstone and shale. The valley fill has an average thickness of 120 feet and ranges in thickness from a feather edge, near the bluff boundaries of the area and along the Chain of Rocks Reach of the Mississippi River, to more than 170 feet near the city of Wood River. The thickness of the valley fill exceeds 120 feet (figure 5) in places near the center of a buried bedrock valley that bisects the area as shown in figure 6.

According to Bergstrom and Walker (1956) recent alluvium makes up the major portion of the valley fill in most of the area. The alluvium is composed largely of fine-grained materials; the grain size increases from the surface down. Recent alluvium rests on older deposits including valley-train materials in many places. The valley-train materials are predominantly medium-to-coarse sand and gravel, and increase in grain size with depth. The coarsest deposits most favorable for development are commonly encountered near bedrock and often average 30 to 40 feet in thickness. Logs of wells in cross section A—A′ in figure 7 and in table 6 show that the valley fill commonly grades from clay to silt to sand and gravel interbedded with layers of silt and clay with increasing depth.

The valley fill is immediately underlain by bedrock formations of Mississippian age in the western part o the area and bedrock formations of Pennsylvanian age in the eastern part of the area. Because of the low permeability of the bedrock formations and poor water quality with depth, the rocks do not constitute an important aquifer in the area.

Soils

The soils of the East St. Louis area were divided into three groups by the University of Illinois Agricultural Experiment Station as follows: bottomland soils, silty terrace soils, and sandy terrace soils. The bottomland soils in St. Clair County were divided into seven soil types by Smith and Smith (1938) as follows: Beaucoup clay loam, Drury fine sandy loam, River sand, Newart silt loam, Gorham clay loam, Dupo silt loam, and Riley fine sandy loam.

Drury fine sandy loam extends in a very narrow strip along the Mississippi River. It is a grayish-yellow to yellow, light brown, medium-to-coarse sand with variable thickness, usually 7 feet. The subsurface and subsoil are not well developed. Surface drainage is slow to rapid and permeability is rapid.

Beaucoup clay loam, Newart silt loam, Gorham cl loam, and Dupo silt loam cover much of the area. They are generally dark gray to grayish brown clay loams to silty clay loams 6 to 15 inches thick. The subsurface var-

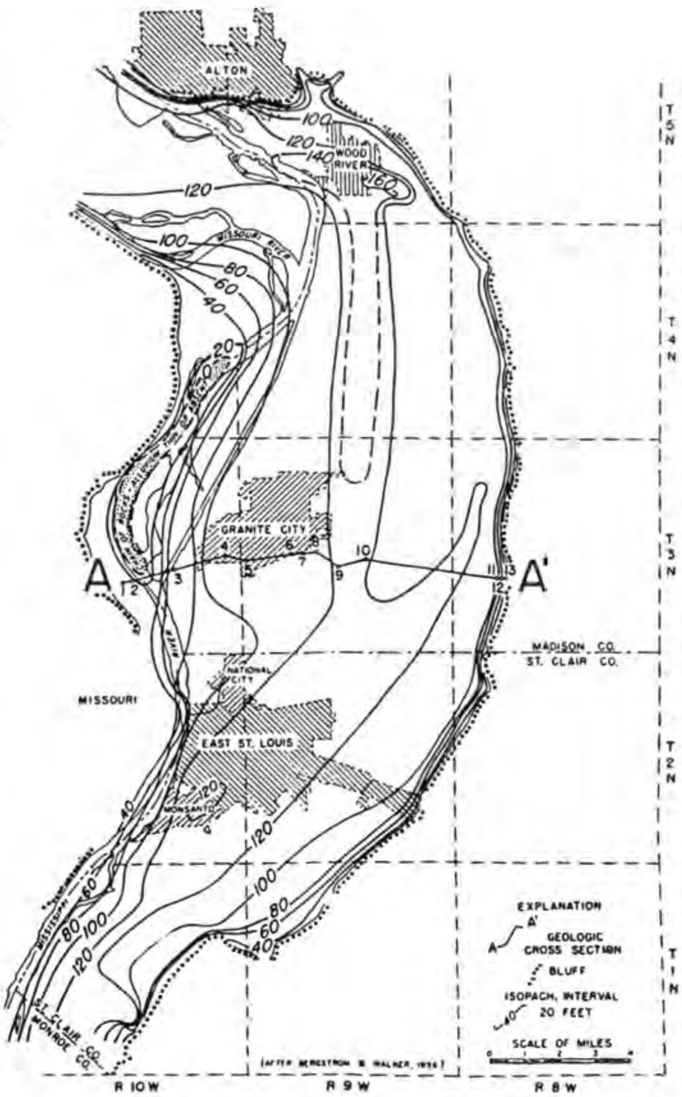


Figure 5. Thickness of the valley fill

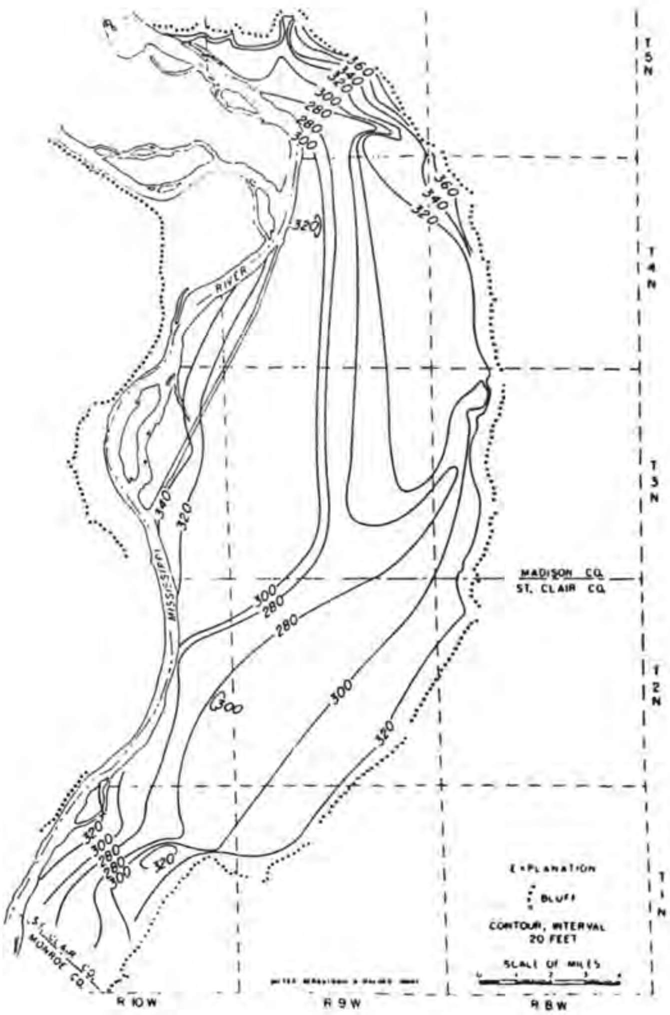


Figure 6. Bedrock topography

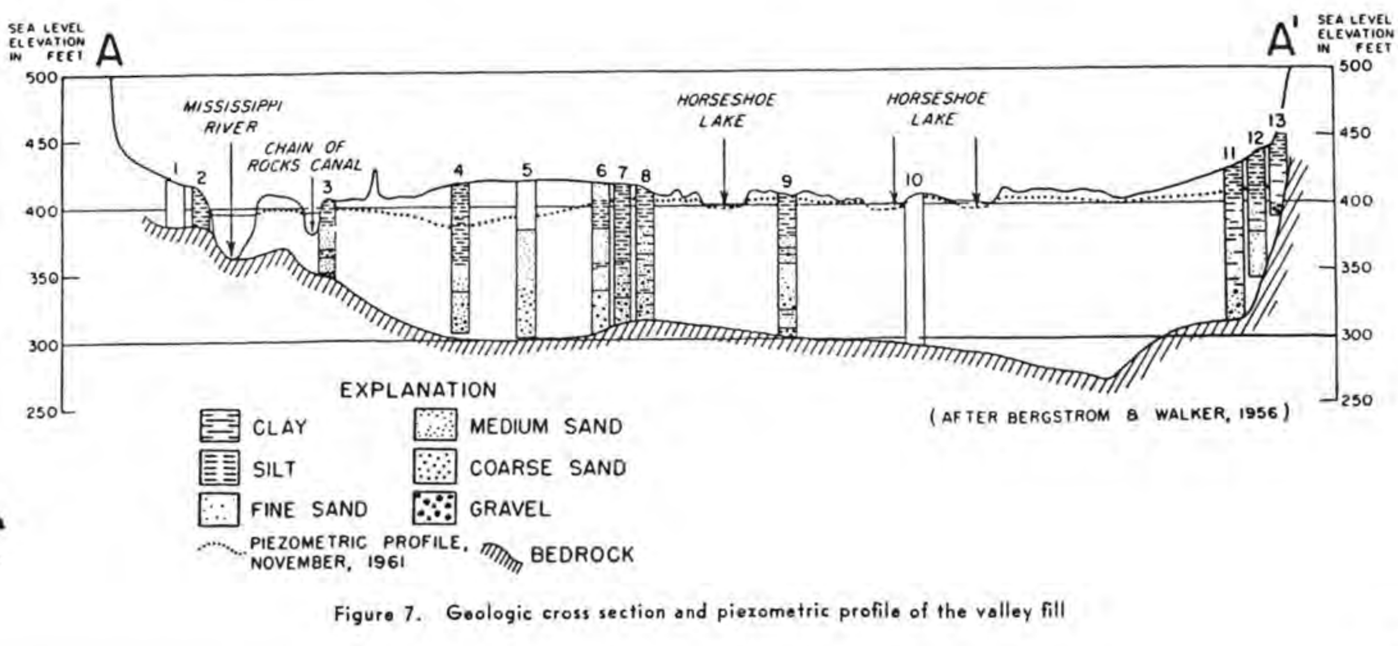


Figure 7. Geologic cross section and piezometric profile of the valley fill

Table 6. Logs of Selected Wells*

Illinois State Geological Survey test hole 3 (1954)—Roxana Water Works, SE ¼ NE ¼ SE¼ SE¼ sec 27, T5N, R9W, Madison Co. Samples studied by R. E. Bergstrom. Est. elev. 445 feet.

	Thickness (ft)	Depth (ft)
Pleistocene Series		
Wisconsin or older Pleistocene		
Clay and silt, yellowish brown, noncalcareous	10	10
Silt and clay, with fine sand, yellowish brown, lumps of pink clay, slightly calcareous	5	15
Sand, fine, dirty, dark reddish brown, calcareous, pink-stained quartz grains	15	30
No samples	5	35
Sand, medium, light reddish brown, calcareous, subrounded grains, rhyolite porphyry, feldspar, gray-wacke, milky chert	15	50
Sand, medium to coarse, as above	20	70
Sand, fine to very coarse, light brown, dirty, gray silt, coal, mica	20	90
Sand, medium to coarse, light reddish brown, subrounded to subangular grains, abundant feldspar, reddish siltstone and rhyolite porphyry	15	105
Sand, coarse to medium, as above	10	115
Sand, very coarse, as above	5	120
Sand, very coarse, with granule gravel, subangular to angular grains, chert, reddish siltstone, granite, gray-wacke	5	127
Pennsylvanian System		
Shale, gray and brown	9.5	136.5

Sinclair Oil Company well 2 (1952)—150 feet N, 1750 feet E of SW corner sec 34, T5N, R9W, Madison Co. Samples studied by R. E. Bergstrom. Est. elev. 431 feet.

	Thickness (ft)	Depth (ft)
Pleistocene Series		
Recent alluvium		
No samples	35	35
Sand, very fine, well sorted, olive gray, mollusk shell fragments, abundant mica, coal, wood	35	70
Silt and clay, with fine sand and small gravel, pebbles to ¾ inch, mollusk shell fragments, calcareous	5	75
Wisconsin or older Pleistocene		
Sand, medium to coarse, yellowish brown, dry sample has pinkish cast, grains subrounded to rounded, slightly calcareous	40	115
Sand and pebble gravel, pebbles to 1.5 inches in diameter, abundant chert, limestone, gray-wacke, rhyolite	7.5	122.5

Illinois Geological Survey test hole 2 (1954)—Lutton farm: 4300 feet S of 80° 32'30" N, 5200 feet E of 90° 15' W, Cahokia Quadrangle, St. Clair Co. Studied by R. E. Bergstrom. Est. elev. 405 feet.

	Thickness (ft)	Depth (ft)
Pleistocene Series		
Recent and older alluvium		
Silt and clay, dark brownish gray	5	5
Silt and clay, with fine sand, dark brownish gray, calcareous, mica	10	15
Sand, fine to medium, dirty, dark olive gray, mica, wood fragments, coal, tiny calcareous spicules, shell fragments	30	45
Sand, coarse to very coarse, with granule gravel, abundant feldspar, granite, gray-wacke, chert, and dolomite granules	30	75
Gravel, granule size, with coarse to very coarse sand, quartz, granite, chert, dolomite granules (driller reports boulders)	20	95
Gravel, granule size with broken limestone rock, chert (pebble count of 50 pebbles—15 gray-wacke and fine-grained basic igneous rock; 12 chert, brown, reddish, and cream-colored; 11 quartz; 3 feldspar; 4 limestone; 4 granite; 1 dolomite); broken rock consists of sharp angular limestone, granite, rhyolite porphyry, and chert	10	105
Broken rock (limestone rubble above solid bedrock?) and granule gravel	7.5	112.5

Union Starch and Refining Company (1952)—950 feet S of 38°42'30" N, 2350 feet E of 90° 10' W, T3N, R10W, Madison Co. Illinois Geological Survey sample set 23406. Studied by R. E. Bergstrom. Est. elev. 422 feet.

	Thickness (ft)	Depth (ft)
Pleistocene Series		
Recent and older alluvium		
Soil, clay, and silt, dark gray	10	10
Sand, fine to coarse, subangular grains, abundant feldspar, tiny calcareous spicules, coal	30	40
Sand, medium, with granule gravel, as above, mollusk shell fragments	10	50
Sand, fine, with granule gravel, poor sorting, calcareous spicules, abundant dark grains of igneous rocks, ferromagnesium minerals, coal	10	60
Gravel, granule size, with coarse sand, granules mainly igneous rocks and feldspar	10	70
No samples	10	80
Sand, medium to fine, calcareous spicules, subangular grains, coal	10	90
No samples	5	95

Table 6 (Continued)

	Thickness (ft)	Depth (ft)
Sand, very coarse to coarse, with granule gravel, pinkish cast, abundant pink-stained quartz grains, subangular to subrounded grains	15	110
Sand, medium, well sorted, pink, subrounded to subangular grains, abundant pink feldspar	5	115

*From Bergstrom and Walker (1956)

ies from silty loam to clay and is generally 2 to 3 feet thick. The subsoil is not well developed. The permeability and surface drainage is generally slow; the permeability of Newart silt loam is moderate.

Riley fine sandy loam covers much of the area near Monsanto, Cahokia, and Centreville. It is a light brown, fine sandy loam 8 to 10 inches thick. The subsurface is a loamy fine sand 8 to 12 inches thick, and the subsoil is a fine sandy loam with occasional clay lenses. Surface drainage is moderate to rapid and permeability is moderately rapid.

Drury fine sandy loam is a brownish yellow to yellowish silt loam to very fine sandy loam and is variable in thickness. It extends along the bluff in strips varying in width from a few feet to several miles. The subsurface is a silt loam to sandy loam about 3 feet thick. The subsoil is not well developed. Surface drainage is rapid and permeability is moderately rapid.

The soils in the East St. Louis area in Madison County have not been divided into soil types. According to McKenzie and Fehrenbacher (1961) bottomland soils predominate; however, silty terrace soils extend in a narrow strip along the bluffs just south of Cahokia Creek to the Madison-St. Clair County line, and in an area that extends from just south of Wood River southeast through Roxana and terminates a few miles southeast of Roxana. Sandy terrace soils extend in a strip a few miles wide from East Alton to Wood River and in a narrow strip southeast of Poag to about 3 miles northwest of Glen Carbon; sandy terrace soils also occur in an area southeast of Roxana.

The bottomland soils in Madison County exhibit a wide range of characteristics similar to those of the soil types in St. Clair County. The silty terrace and sandy terrace soils have moderately good to good drainage and moderately rapid to rapid permeability.

Occurrence of Ground Water

Ground water in the valley fill occurs under leaky artesian and water-table conditions. Leaky artesian conditions exist at places where fine-grained alluvium, consisting of silt and clay with some fine sand that impedes or retards the vertical movement of water, overlies coarser alluvium and valley-train deposits; water in these deposits is under artesian pressure. Under leaky artesian conditions, water levels in wells rise above the top of the valley-train and coarse alluvium deposits to stages within the finer grained alluvium. Water-table conditions prevail at many places where alluvium is missing and the upper surface of the zone of saturation is in valley-train deposits or the coarser alluvium, and at places within deep cones of depression created by heavy pumping where water levels in wells rise to stages within the valley-train deposits or the coarser alluvium and water is unconfined.

As shown in figure 8, leaky artesian conditions prevail in most of the area. Water-table conditions prevail in a wide belt from East Alton through Poag where alluvium is missing and heavy pumping in the vicinity of Wood River has lowered water levels below the base of the finer grained alluvium. Water-table conditions also prevail in: 1) the Monsanto and National City areas where heavy pumping has lowered water levels to stages within the valley-train deposits and coarser alluvium; 2) an area

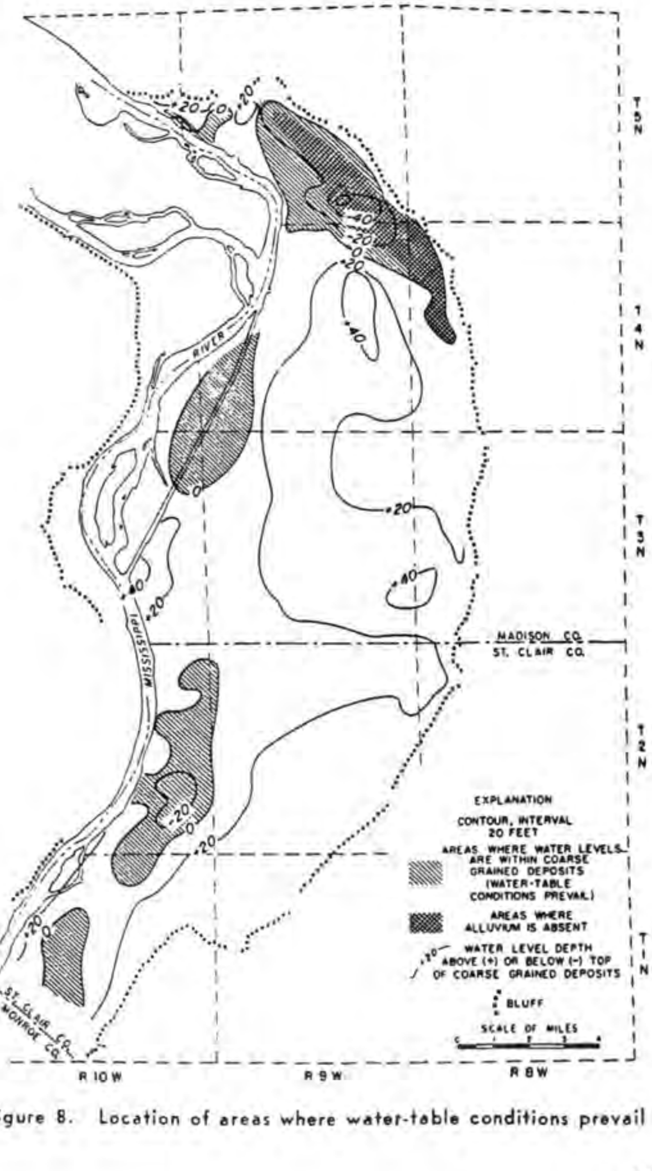


Figure 8. Location of areas where water-table conditions prevail

through Dupo and along the northern reach of the Chain of Rocks Canal where the finer grained alluvium is thin and water levels are in the coarser deposits; and 3) locally in the vicinity of well fields in the Granite City area and other areas where the saturated thickness of the finer grained alluvium is not great. The saturated thickness of the finer grained alluvium is greatest west of Poag near the center of T4N R9W, along the Mississippi River near Venice, and in an area 4 miles northwest of Collinsville.

Because water occurs most commonly under leaky artesian conditions, the surface to which water rises, as defined by water levels in wells, is hereafter called the piezometric surface.

HYDRAULIC PROPERTIES

The principal hydraulic properties of the valley fill and alluvium influencing water-level declines and the yields of wells in the East St. Louis area are the coefficients of transmissibility, or permeability, and storage. The capacity of a formation to transmit ground water is expressed by the coefficient of transmissibility, T, which is defined as the rate of flow of water in gallons per day, through a vertical strip of the aquifer 1 foot wide and extending the full saturated thickness under a hydraulic gradient of 100 percent (1 foot per foot) at the prevailing temperature of the water. The coefficient of transmissibility is the product of the saturated thickness of the aquifer, m, and the coefficient of permeability, P, which is defined as the rate of flow of water in gallons per day, through a cross-sectional area of 1 square foot of the aquifer under a hydraulic gradient of 100 percent at the prevailing temperature of the water. The storage properties of an aquifer are expressed by the coefficient of storage, S, which is defined as the volume of water released from storage per unit surface area of the aquifer per unit change in the water level.

Aquifer Tests

The hydraulic properties of the valley fill and alluvium may be determined by means of aquifer tests, wherein the effect of pumping a well at a known constant rate is measured in the pumped well and at observation wells penetrating the aquifer. Graphs of drawdown versus time after pumping started, and/or drawdown versus distance from the pumped well, are used to solve equations which express the relation between the coefficients of transmissibility and storage and the lowering of water levels in the vicinity of a pumped well.

The data collected during aquifer tests can be analyzed by means of the nonequilibrium formula (Theis, 1935). Further, Walton (1962) describes a method for applying the Theis formula to aquifer test data collected under water-table conditions, and gives equations for compensating observed values of drawdown for decreases in the saturated thickness of an aquifer.

Six controlled aquifer tests were made during the period 1952 to 1962. The results of the tests are summarized in table 7.

Table 7. Results of Aquifer Tests

Owner	Location of test site	Date of test	Duration of test *(days)*	Pumping rate *(gpm)*	Coefficient of transmissibility *(gpd/ft)*	Saturated thickness *(ft)*	Coefficient of permeability *(gpd/sq ft)*	Coefficient of storage	Method of analysis*
Olin Mathieson Chemical Corp.	Madison County, T5N, R9W, sec 19	May 29-Jun 1, 1956	3	760	95,600	90	1060	0.135	D-D
City of Wood River	Madison County, T5N, R9W, sec 28	Nov 20-21, 1962	1	491	134,000	60	2240	0.155	D-D
Shell Oil Co.	Madison County, T5N, R9W, sec 33	Mar 3-6, 1952	3	510	210,000	100	2100	0.002	D-D
Southwestern Campus of IU, Edwardsville	Madison County, T4N, R8W, sec 20	Dec 13-17, 1960	4	308	131,000	84	1560	0.020	T-D
Mobil Oil Co.	St. Clair County, T2N, R10W, sec 25	Oct 25-26, 1961	1	630	212,000	73	2900	0.100	T-D
Monsanto Chemical Corp.	St. Clair County, T2N, R10W, sec 27	Aug 4-8, 1952	4	1100	210,000	75	2800	0.082	T-D

*D-D, distance-drawdown; T-D, time-drawdown

An aquifer test was made October 25 and 26, 1961, the Mobil Oil Company Refinery near Monsanto by the State Water Survey in cooperation with the company. The test site was located in an area about 2600 feet north and 3500 feet west of the intersection of T2N, R10W and T1N, R9W. The effects of pumping well 19 were measured in test well 8, well 6, and well 20. The locations of wells used in the test (test 1) and test wells for which drillers logs are available are shown in figure 9. Pumping was

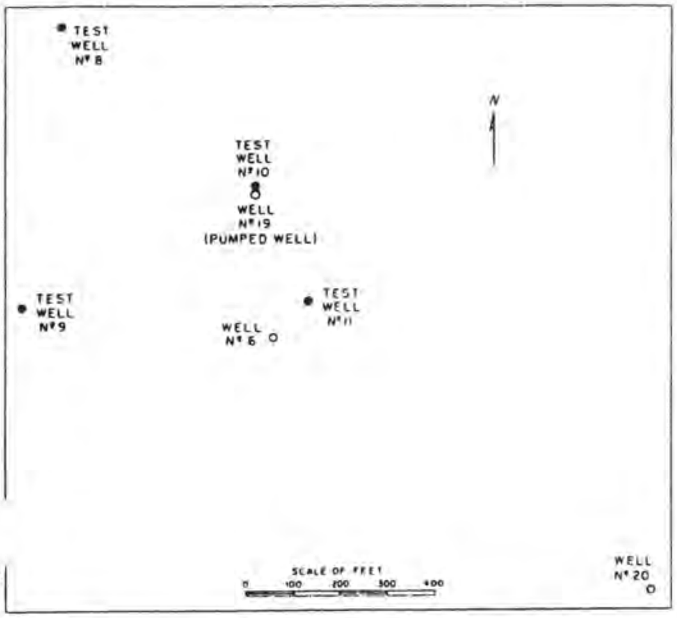


Figure 9. Location of wells used in aquifer test 1

started at 9 a.m. October 25 and continued for 24 hours at a constant rate of 630 gpm. Pumping was stopped at 9 a.m. October 26 and water levels were allowed to recover for 1 hour, after which a step-drawdown test was conducted. Water levels were measured continuously with a recording gage in well 6, and periodically with a steel tape in well 20 and test well 8.

Well 19 is 16 inches in diameter, was drilled to a depth of 114 feet, and is equipped with 35 feet of No. 50 continuous slot Johnson Everdur screen between the depths of 79 and 114 feet. The well is an artificial pack well with a pack thickness of about 9 inches. Well 6 is 16 inches in diameter, 115 feet deep, and is screened at the bottom with 30 feet of 16-inch diameter Johnson Everdur screen with varying continuous slot sizes of 40, 50, 70, and 90. The thickness of the pack is not known. Well 20 is 24 inches in diameter and is 107 feet deep; there is 35 feet of 24-inch diameter Johnson Everdur screen at the bottom. The lower 17.5 feet of the screen is No. 100 slot and the upper 17.5 feet is No. 60 slot. The pack thickness is 9 inches. Test well 8 is 8 inches in diameter and 105 feet deep. The screen and casing are constructed of wood. The screen is 53 feet long with $\frac{3}{16}$ by 3-inch slots. The thickness of the pack is 5 inches. The logs of wells are given in table 8.

A time-drawdown field data graph (figure 10) for well 6 was superposed on the nonequilibrium type curve devised by Theis and described by Jacob (1940). The Theis (1935) nonequilibrium equations were used to determine coefficients of transmissibility and storage of the aquifer for data on the first and third segments of the time-drawdown graph. The coefficient of storage computed from the first segment of the time-drawdown curve is in the artesian range and cannot be used to predict long-term declines of the water table. The coefficient of storage (0.10) computed from the third segment is in the water-table range. The coefficient of transmissibility computed from the third segment is 212,000 gpd/ft.

An aquifer test (test 2) was made December 13-17, 1960, by Warren and Van Praag, Inc., Layne-Western Company, and the State Water Survey in cooperation with the Southwestern Campus of Southern Illinois University near Edwardsville. The test site is located west of Edwardsville in section 20, T4N, R8W. Three wells as shown in figure 11 were used. Pumping was started at 1:45 p.m. December 13, and was continued at a constant rate of 308 gpm until 12:30 p.m. December 17. Pumping was then stopped and water levels were allowed to recover for 1 hour. At 1:30 p.m. pumping was resumed at successive rates of 200, 300, 400, and 500 gpm, each maintained for 30 minutes. Water levels were measured periodically in the observation wells and pumped well during the test.

Observation well 1 was 2 inches in diameter and 94 feet deep, and the bottom 5 feet of pipe was slotted. Observation well 2 was 2 inches in diameter, 89 feet deep, and the bottom 6 feet of pipe was slotted. The pumped well was 10 inches in diameter and was drilled to a depth of 95 feet; 20 feet of screen was installed at the bottom. The well was an artificial pack well with a pack thickness of 3.5 inches. Logs of wells are given in table 9.

A time-drawdown field data graph (figure 12) for observation well 2 was superposed on the nonequilibrium type curve. The Theis (1935) equations were used to determine coefficients of transmissibility and storage of the aquifer for data on the third segment of the time-drawdown curve. The coefficient of transmissibility was computed to be 131,000 gpd/ft. The coefficient of storage (0.020) is in the water-table range.

An aquifer test (test 3) was made November 20 and 21, 1962, by Warren and Van Praag, Inc., Layne-Western Company, and the State Water Survey in cooperation with the city of Wood River. The test site was located in sec. 28, T5N, and R9W. Six wells as shown in figure 13 were used. Pumping was started at 9:45 a.m. November 20 and was continued at a constant rate of 491 gpm until 8:15 a.m. November 21. Pumping was then stopped and water levels were allowed to recover for 50 minutes. At 9:10 a.m. pumping was resumed and a step-drawdown test was conducted. Recording gages were installed in

Table 8. Drillers Logs of Wells Used in Aquifer Test 1

Formation	From (ft)	To (ft)
Test Well 8		
Clay fill	0	7
Fine sand	7	24
Fine to medium gray sand	24	37
Medium to coarse sand	37	41
Fine sand	41	55
Medium to coarse sand	55	64
Medium sand	64	65
Medium to coarse sand	65	73
Very coarse sand	73	80
Coarse to medium sand with cobbles	80	84
Coarse to medium sand	84	85
Medium to fine sand with gravel	85	90
Medium sand with gravel	90	95
Fine sand with gravel at 103.5 feet	95	103.5
Fine to coarse sand with gravel and cobbles	103.5	104
Coarse sand to heavy gravel with cobbles	104	106.9
(unable to drill beyond 106.9 feet because of heavy cobbles)		

Formation	From (ft)	To (ft)
Test Well 9		
Fill, clay, gravel	0	5
Silt and sandy silt	5	33
Medium gray sand	33	40
Fine sand, gray	40	45
Medium sand, gray	45	50
Coarse sand, gray, trace of clay	50	52
Very coarse sand with gravel	52	56
Medium coarse sand with gravel	56	58
Coarse gravel sand with gravel	58	62
Very coarse sand with gravel	62	63
Coarse sand and very coarse sand	63	72
Coarse to medium sand	72	79
Fine to medium sand with cobbles	79	79.5
Fine sand	79.5	80
Fine to medium sand	80	82
Medium coarse sand with gravel	82	83
Very coarse sand with gravel and lignite, cobbles at 88 feet	83	89.5
Coarse to medium sand	89.5	98
Very fine sand	98	100
Fine sand with gravel, cobbles at 102 feet	100	102.5
Coarse sand with gravel	102.5	104
Coarse sand with gravel and cobbles	104	113

Formation	From (ft)	To (ft)
Test Well 10		
Gravel fill, gumbo	0	10
Dark silt	10	32
Fine gray sand	32	33
Dark fine silt	33	36
Medium fine sand	36	43
Fine sand	43	47
Test Well 10 (Continued)		
Dark gray silty sand	47	55
Fine sand	55	57
Medium fine sand	57	74
Very coarse sand with pea gravel and lignite	74	74.5
Very coarse sand with cobbles	74.5	80
Very coarse sand	80	87
Medium coarse sand with cobbles from 89 to 91 feet	87	90
Fine sand	90	95
Very fine sand	95	99
Very fine sand with cobbles at 100.5 feet	99	100
Coarse sand with cobbles	100	102
Coarse sand with cobbles	102	103
Coarse sand	103	104
Coarse sand with gravel	104	114

Formation	From (ft)	To (ft)
Test Well 11		
Mixture of clay, fill, silt, fly ash	0	10
Fine gray silt	10	15
Very fine gray sand	15	20
Fine gray sand	20	25
Fine to medium gray sand	25	40
Medium gray sand	40	45
Medium to fine gray sand	45	52
Medium to coarse sand	52	55
Fine to medium sand	55	61
Medium to fine sand	61	73
Medium sand	73	76
Coarse sand	76	80
Very coarse sand with cobbles	80	87
Medium to coarse sand with cobbles	87	95
Fine to medium sand	95	95.5
Medium to coarse sand with ½-inch gravel	95.5	100
Very coarse sand and gravel with boulders	100	115

Formation	From (ft)	To (ft)
Well 20		
Silty sandy gray clay	0	15
Medium gray sand	15	20
Fine gray sand	20	25
Coarse gray sand	25	30
Fine to medium gray sand	30	35
Very fine sand	35	53
Very coarse sand with 2-inch gravel	53	55
Medium sand with gravel	55	57.5
Medium to coarse sand with gravel	57.5	60
Very coarse sand with pea gravel	60	65
Medium to coarse sand with gravel	65	75
Coarse sand	75	80
Very coarse sand with pea gravel	80	83.5
Medium to coarse sand	83.5	85
Very coarse sand with 2-inch gravel	85	107

elief wells 137 and 139. Water levels were measured periodically with a steel tape in the pumped well, test hole 5, test hole 4, and relief well 140.

The pumped well was 10 inches in diameter and was drilled to a depth of 84 feet; 20 feet of 8-inch slotted pipe was installed at the bottom. The well is an artificial pack well with a pack thickness of 4 inches. Test holes 4 and 5 were 2 inches in diameter and 70 and 66.5 feet in

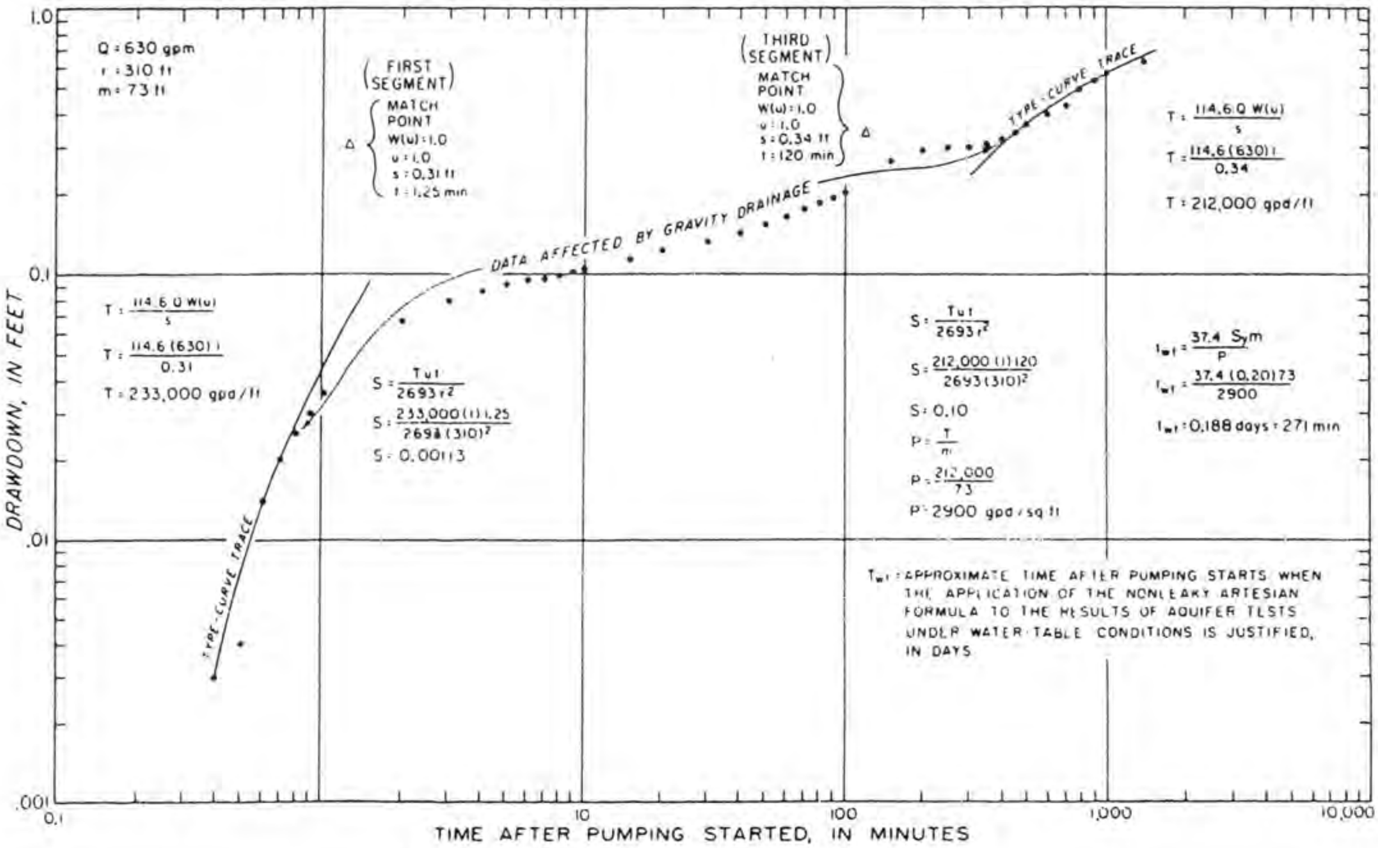


Figure 10. Time-drawdown data for well 6, aquifer test 1

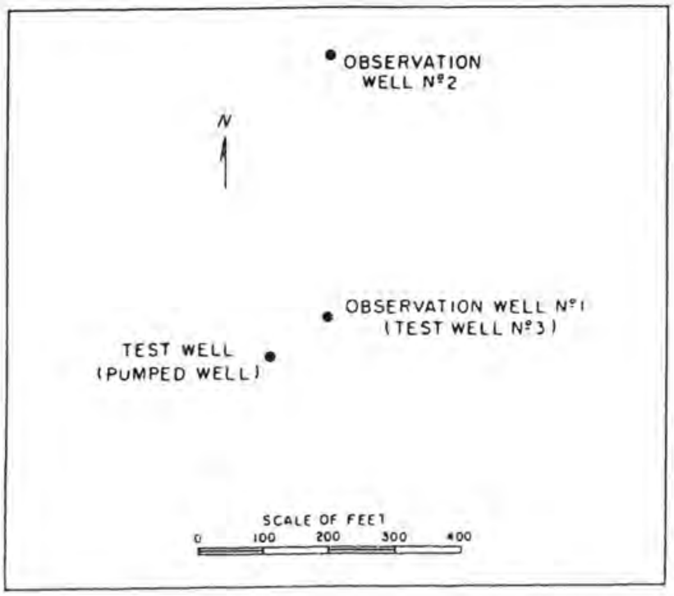


Figure 11. Location of wells used in aquifer test 2

Table 9. Drillers Logs of Wells Used in Aquifer Test 2

Formation	From (ft)	To
Test Well (Pumped Well)		
Sandy clay	0	14
Fine brown sand	14	50
Coarse gray sand	50	75
Fine-to-medium brown sand	75	90
Medium gray sand	90	95
Fine brown sand	95	98
Observation Well 1		
Brown clay	0	14
Fine brown sand, clay streaks	14	50
Medium gray sand, loose	50	75
Coarse gray sand, some gravel, loose	75	90
Fine sand	90	100
Light gray shale	100	130
Limestone		130
Observation Well 2		
Brown clay	0	14
Fine red sand, clay streaks	14	65
Medium gray sand, little gravel, few clay balls	65	90
Fine sand	90	100

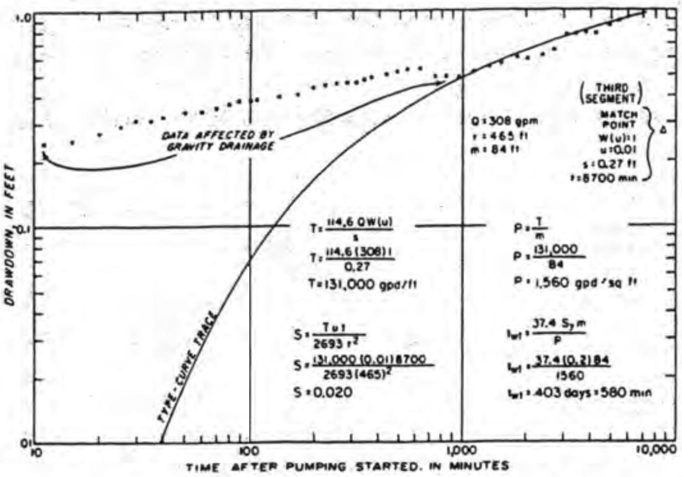


Figure 12. Time-drawdown data for observation well 2, aquifer test 2

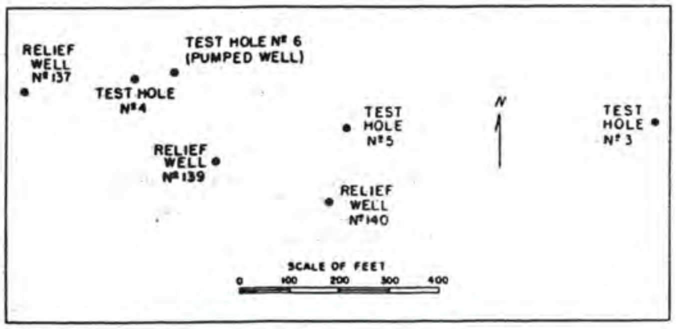


Figure 13. Location of wells used in aquifer test 3

depth, respectively. The lower 6.4 feet of casing in each test hole was slotted. The logs of test holes are given in table 10.

A distance-drawdown field data graph (figure 14) prepared with water-level data collected in the observation wells after a pumping period of 1335 minutes was superposed on the nonequilibrium type curve. The Theis (1935) equations were used to determine coefficients of transmissibility and storage of the aquifer. The coefficient of transmissibility was computed to be 134,000 gpd/ft. The coefficient of storage (0.155) is in the water-table range.

The cone of depression created by pumping a well near a river that is hydraulically connected to the aquifer is distorted. The hydraulic gradients between the river and the pumped well will be steeper than the hydraulic gradients on the land side of the well. The flow towards the well will be greatest on the river side of the well, and under equilibrium conditions most of the pumped water will be derived from the river.

When the well is pumped, water is initially withdrawn from storage within the aquifer in the immediate vicinity of the well. If pumping is continued long enough water levels in the vicinity of the river will be lowered and water that under natural conditions would have discharged into the river as ground-water runoff or into the atmosphere as evapotranspiration is diverted toward the pumped well. Water levels are ultimately lowered below all or part of the river bed in the immediate vicinity of the well, and the aquifer is then recharged by the influent seepage of surface water. The cone of depression will continue to grow until sufficient area of the river bed

Table 10. Drillers Logs of Test Holes Used in Aquifer Test 3

Formation	From (ft)	To (ft)
Test hole 3		
Brown clay	0	20
Soft blue clay	20	46
Fine sand	46	50
Medium to coarse	50	82
Sand, loose	82	104
Gray clay	104	116
Fine sand, loose	116	120
Red clay	120	
Rock		
Test hole 4		
Brown clay	0	9
Fine sand, clay streaks	9	25
Medium sand, some clay	25	30
Fine tight sand	30	52
Coarse sand and gravel, loose	52	79
Hard gray clay	79	83
Fine sand, clay streaks	83	90
Bedrock		90
Test hole 5		
Brown clay	0	11
Fine sand and clay	11	17
Fine sand	17	55
Coarse sand and gravel, loose	55	83
Gray clay	83	100
Test hole 6 (Pumped Well)		
Brown clay	0	10
Fine sand and clay	10	18
Fine sand	18	48
Coarse sand and gravel, boulders drilled like rock ledge at 57 feet	48	80
Gray clay	80	84

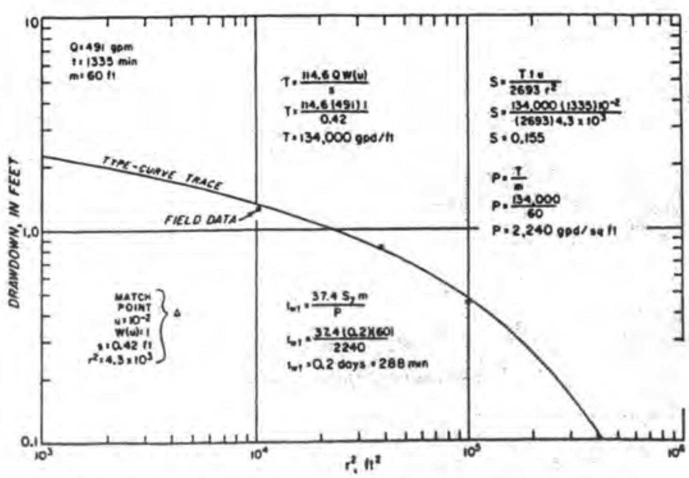


Figure 14. Distance-drawdown data for aquifer test 3

intercepted and the cone is deep enough so that the induced infiltration balances discharge.

The area of the river bed over which recharge takes place is replaced by a line source. According to the image well theory (Ferris, 1959), the effect of a line source on the drawdown in an aquifer, as a result of pumping from a well near the line source, is the same as though the aquifer were infinite and a like recharging well were located across the line source, and on right angles thereto, and at the same distance from the line source as the real pumping well. Based on the image well theory and the nonequilibrium formula, the drawndown distribution in an aquifer bounded by a line source under equilibrium conditions is given by the following equation:

$$s = [528Q \log_{10} (r_i/r_p)]/T \tag{1}$$

where:

s = drawdown at observation point, in ft

Q = discharge of pumped well, in gpm

r_i = distance from image well to observation point, in ft

r_p = distance from pumped well to observation point, in ft

T = coefficient of transmissibility, in gpd/ft

In terms of the distance between the pumped well and the line source or recharge boundary, equation 1 was expressed by Rorabaugh (1956) as

$$s = [528Q \log_{10} (\sqrt{4a^2 + r_p^2 - 4a\, r_p \cos \Phi}/r_p)]/T \tag{2}$$

where:

a = distance from pumped well to recharge boundary, in ft

Φ = angle between a line connecting the pumped well and the image well and a line connecting the pumped well and the observation point

For the particular case where the observation well is on a line parallel to the recharge boundary, equation 2 may be written as follows:

$$s = [528Q \log_{10} (\sqrt{4a^2 + r_p^2}/r_p)]/T \tag{3}$$

Equations 1 through 3 assume that the cone of depression has stabilized, water is no longer taken from storage within the aquifer, and equilibrium conditions prevail. The pumping period required to stabilize water levels can be computed by using the following equation (see Foley, Walton, and Drescher, 1953):

$$t_e = 3.26a^2s/[T\varepsilon \log_{10} (2a/r_p)^2] \tag{4}$$

where:

t_e = time after pumping starts before equilibrium conditions prevail, in days

s = coefficient of storage, fraction

ε = deviation from absolute equilibrium (arbitrarily assumed to be 0.05)

In many cases the stabilization of the cone of depression can be attributed either to the effects of slow gravity drainage, effects of leakage through a confining bed (Walton, 1960a), or effects of induced infiltration if the effects of partial penetration are excluded. Walton (1963) gave methods for proving whether or not water levels stabilize because of the effects of induced infiltration.

According to Walton (1963) the coefficient of transmissibility can often be determined from distance-drawndown data for observation wells on a line parallel to the recharge boundary. Provided the wells are not too distant from the pumped well and not too close to the recharge boundary, the effects of induced infiltration on drawdowns in the wells is approximately equal because the wells are for practical purposes equidistant from the image well associated with the recharge boundary. A plot of maximum drawdowns in the observation wells versus the logarithm of distance from the pumped well will yield a straight-line graph. The slope of the straight line is substituted in the following equation (Cooper and Jacob, 1946) to compute the coefficient of transmissibility:

$$T = 528Q/\Delta s \tag{5}$$

where:

T = coefficient of transmissibility, in gpd/ft

Q = discharge of pumped well, in gpm

Δs = drawdown difference per log cycle as determined from distance-drawdown graph, in ft

If T is known, the distance from the pumped well to the recharge boundary, a, can be computed with maximum drawdowns in each observation well on a line parallel to the stream and the following equation:

$$\log_{10} \sqrt{4a^2 + r_p^2}/r_p = Ts/528Q \tag{6}$$

where:

s = drawdown, in ft

a = distance from pumped well to recharge boundary, in ft

r_p = distance from pumped well to observation well, in ft

Q = discharge of pumped well, in gpm

T = coefficient of transmissibility, in gpd/ft

The maximum drawdowns in the observation wells are much less because of the effects of recharge than they would be if the aquifer were infinite; thus, the coefficient of storage cannot be determined from the distance-drawdown graph.

The nonequilibrium formula (Theis, 1935) and computed values of T and a can be used to determine the coefficient of storage. Several values of the coefficient of storage are assumed, and maximum drawdowns in each observation well are computed taking into consideration the effects of the image well associated with the recharge boundary and the pumped well. The computed drawdowns in each observation well are then compared with actual drawdowns, and the coefficient of storage that provided computed drawdowns equal to actual drawdowns is assigned to the aquifer.

Three aquifer tests under induced infiltration conditions were made during the period 1952 to 1956. The results of the tests are summarized in table 7.

An aquifer test (test 4) was made March 3-6, 1952, on property owned by the Shell Oil Company along the Mississippi River in sec. 33, T5N, R9W. The test was conducted for the Shell Oil Company by Ranney Method Water Supplies, Inc. Seven wells, grouped as shown in figure 15, were used. Four wells were approximately

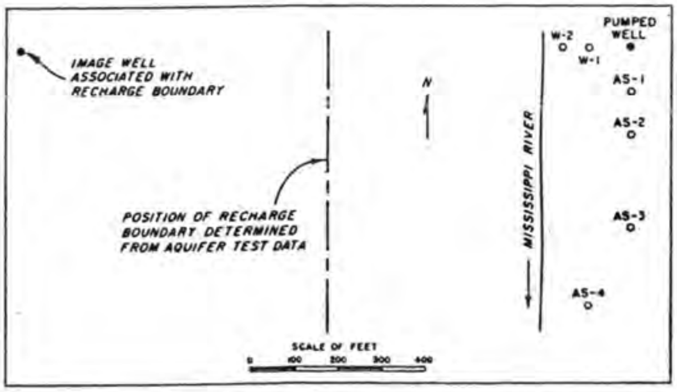


Figure 15. Location of wells used in aquifer test 4

parallel to and about 200 feet east of the Mississippi River. Pumping was started at 9:25 a.m. and was continued at a constant rate of 510 gpm for three days. Pumping was stopped at 9:25 a.m. March 6, and water levels were allowed to recover.

Observation wells AS-1, AS-2, and AS-3 were reported to be 7 inches in diameter and averaged 60 feet in depth; wells AS-4, W-1, and W-2 were 7 inches in diameter and were drilled to depths of 119, 112, and 55 feet respectively. The pumped well was 12 inches in diameter and 100 feet deep. Data on lengths of screens were not available. Recording gages were installed on the six observation wells and the Mississippi River. Logs of wells used in the test are given in table 11.

Values of drawdown in wells AS-1, AS-2, and AS-3 at a time 1800 minutes after pumping started were plotted on semilogarithmic paper against values of distance from the pumped well as shown in figure 16. A straight line was drawn through the points. The slope of the straight line per log cycle and the pumping rate were substituted into equation 5 and the coefficient of transmissibility was computed to be 210,000 gpd/ft.

The distance from the pumped well to the recharge boundary was determined by substituting the computed value of T, the measured rate of pumping, and values of drawdowns in the observation wells into equation 6 and solving for the distance a. The average distance a was found to be about 700 feet.

The coefficient of storage was determined to be 0.002 by using the computed values of T, a, the drawdowns in observation wells, and the nonequilibrium formula. Fine-grained alluvial deposits (see table 11) occur in the portion of the aquifer unwatered by pumping.

An aquifer test (test 5) was made May 29 through June 1, 1956, by Ranney Method Water Supplies, Inc., for the Olin-Mathieson Chemical Corporation. E. G. Jones, Water Survey field engineer, assisted in making the test. The test site was just southeast of the confluence of Wood River and the Mississippi River in sec. 19, T5N, R9W. Eight wells, grouped as shown in figure 17 were used. The wells were arranged in a "T" pattern with four wells parallel to and 350 feet north of the Mississippi River. Pumping was started at 1:30 p.m. on May 29 and stopped at 1:30 p.m. on June 1. The pumping rate during the test was held constant at a rate of 760 gpm.

Table 11. Drillers Logs of Wells Used in Aquifer Test 4

Formation	From (ft)	To (ft)
Well AS-1		
Brown silty sand	0	19
Blue clay	19	33
Fine gray sand	33	41
Coarse sand and sand and small gravel	41	60
Well AS-2		
Brown silty clay	0	19
Blue clay	19	32
Fine gray sand	32	42
Coarse gravel and small and medium gravel	42	62
Well AS-3		
Brown silty clay	0	19
Blue clay	19	34
Fine gray sand	34	42
Coarse gravel and small and medium gravel	42	60
Well AS-4		
Brown clay	0	5
Dirty fine gray sand	5	37
Fine gray sand	37	51
Coarse sand and gravel	51	71
Fine red sand	71	92
Medium sand and gravel	92	112
Medium sand and gravel	112	119
Well W-1		
Brown clay	0	4
Soft blue clay	4	26
Fine sand	26	37
Sand and gravel	37	116
Hard blue clay	116	118
Bedrock		
Well W-2		
Clay	0	3
Gray silt	3	28
Fine gray sand	28	40
Coarse sand and gravel	40	55

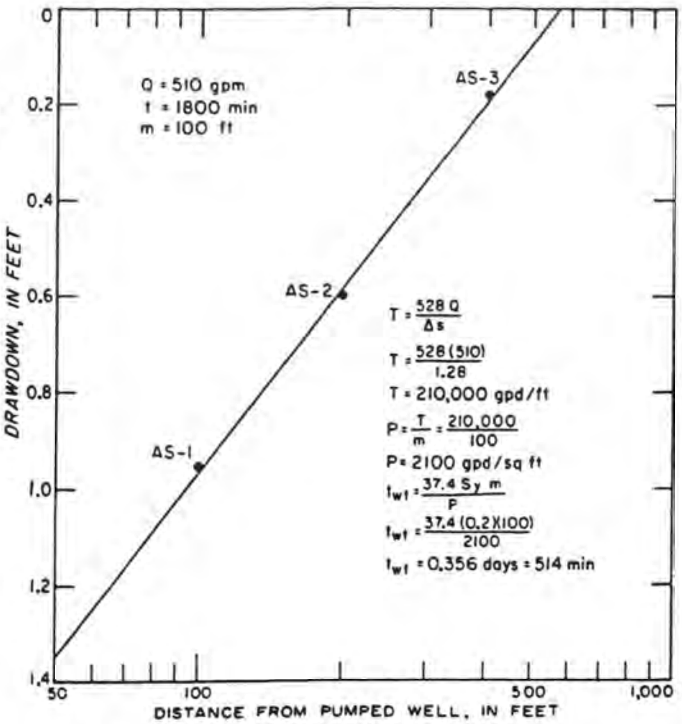


Figure 16. Distance-drawdown data for aquifer test 4

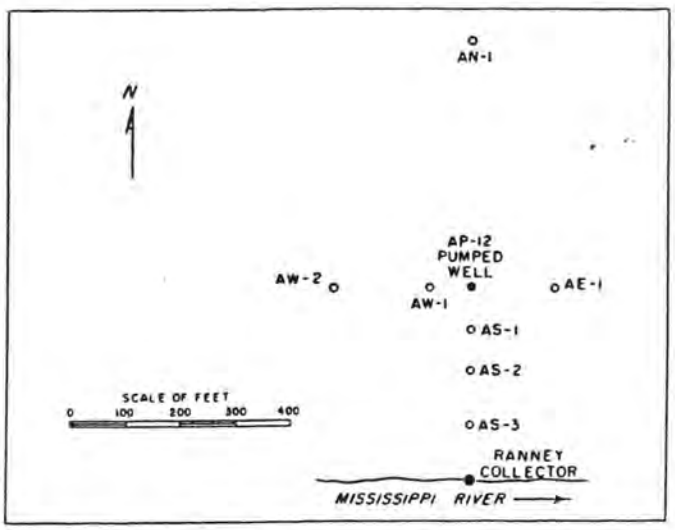


Figure 17. Location of wells used in aquifer test 5

The pumped well was 12 inches in diameter and 88 feet deep; the lower 10 feet of the well was screened. Observation wells AS-1, AS-2, AN-1, AW-1, AW-2, and AE-1 were 6 inches in diameter and averaged about 90 feet in depth. Well AE-3 was 6 inches in diameter and 124 feet in depth. Drillers logs of wells are given in table 12. Recording gages were installed on the observation wells and the Mississippi River. Values of drawdown in wells AS-1, AW-1, AE-1, AS-2, AS-3, and AW-2 at a time 1830 minutes after pumping started were plotted on semilogarithmic paper against values of distances from the pumped well as shown in figure 18. A straight line was drawn through the points. The slope of the straight line per log cycle and the pumping rate were substituted into equation 5 and the coefficient of transmissibility was computed to be 95,600 gpd/ft. The slope of the straight line per log cycle from distance-drawdown data on a line perpendicular to the river and on a line parallel to the river are approximately the same suggesting that the effects of induced infiltration on drawdowns were negligible. The coefficient of storage, S, was computed from the following equation (Cooper and Jacob, 1946):

$$S = Tt/4790r_o^2 \qquad (7)$$

where:

S = coefficient of storage, fraction
t = time after pumping started, in min
T = coefficient of transmissibility, in gpd/ft
r_o = intercept of straight line with zero drawdown axis, in ft

The coefficient of storage (0.135) is in the water-table range.

The distance a was found to be 100 feet from the river's edge, as determined from water-level data collected during a production test February 13-19, 1959, using the collector well constructed at the site of aquifer test 5, hydraulic properties of the aquifer determined from the aquifer test May 29 - June 1, 1956, and equation 6. Pumping from the collector well was started at 8 a.m. on February 13 and continued at a constant rate of 7000 gpm until 3:15 p.m. February 17 when the pumping rate was increased to 8400 gpm. The pumping test continued at a rate of 8400 gpm until 8:15 p.m. February 19 when pumping was stopped and water levels were allowed to recover. Recording gages were installed on observation wells AS-3, AE-1, and AN-1. Frequent water-level measurements were made with a steel tape in well AS-2. In addition, recording gages were installed on the Mississippi River, on the collector well, and on an observation well immediately outside the collector well.

An aquifer test (test 6) was made August 4-8, 1952, by Ranney Method Water Supplies, Inc., for the Monsanto Chemical Corporation. The test site is located east of Monsanto, along the Mississippi River in sec. 27, T2N, R10W. Seven wells, grouped as shown in figure 19 were used. The wells were arranged in a 'T' pattern with four wells parallel to and 515 feet east of the Mississippi River and three wells perpendicular to the river. Pumping was started at 6 p.m. August 4 and was continued at a constant rate of 1100 gpm until 6 p.m. August 8 when pumping was stopped and water levels were allowed to recover.

Observation wells S-1, W-1, N-1, S-2, W-2, and W-3 were 7 inches in diameter and were drilled to depths of about 100 feet. The pumped well was 12 inches in diameter and was drilled to a depth of 99 feet; 10 feet of screen was installed at the bottom. Available logs of wells are given in table 13. Recording gages were installed on the

Table 12. Drillers Logs of Wells Used in Aquifer Test 5

Formation	From (ft)	To (ft)
Well AP-12 (Pumped Well)		
Fine brown sand, silty	0	15
Fine brown sand, silty, scattered gravel	15	28
Medium to pea gravel, fine sand with scattered clay balls, gray	28	40
Fine sand, scattered gravel	40	60
Very fine sand	60	78
Medium to coarse gravel, fine sand with scattered clay balls	78	81
Medium to pea gravel, medium sand	81	85
Medium to pea gravel, coarse sand	85	88
Gray clay	88	(Total depth)
Well AS-1		
Fine brown sand, silty	0	27
Fine sand, scattered gravel, clay balls	27	30
Medium to pea gravel, fine sand, clay	30	37
Very fine gray sand	37	73
Medium to pea gravel, medium to coarse sand	73	89
Clay balls	89	
Well AS-2		
Fine brown sand, silty	0	28
Fine brown sand, clay balls	28	30
Very fine gray sand	30	37
Medium to coarse gravel, fine sand	37	73
Medium to pea gravel, fine sand	73	89
Clay balls	89	
Well AS-3		
Very fine brown sand, silty	0	22
Medium to pea gravel, fine sand	22	34
Fine gray sand	34	70
Medium to pea gravel, fine sand	70	75
Medium to pea gravel, medium sand	75	90
Gray clay	90	96
Medium sand, scattered gravel, clay balls	96	110
Medium sand, scattered pea gravel	110	124
Sandstone rock	124	
Well AN-1		
Fine sand, brown, silty	0	25
Fine gray sand	25	35
Medium sand, scattered gravel	35	56
Medium sand, scattered gravel, clay balls	56	59
Medium sand, scattered gravel	59	72
Medium to pea gravel, coarse sand	72	80
Medium to pea gravel, medium sand	80	82
Clay balls and boulders	82	83
Medium to fine sand, scattered gravel, clay balls	83	89
Well AW-1		
Fine brown sand, silty	0	20
Medium sand, clay balls	20	31
Fine gray sand, scattered gravel	31	37
Very fine gray sand	37	76
Medium to pea gravel, medium sand	76	87
Gray clay	87	88
Well AW-2		
Fine brown sand, silty	0	38
Very fine gray sand	38	55
Very fine gray sand, scattered gravel	55	57
Very fine gray sand	57	84
Medium to fine sand, scattered gravel	84	89
Clay balls	89	
Well AE-1		
Fine brown sand, silty	0	28
Fine gray sand, clay balls	28	32
Very fine sand	32	75
Medium to pea gravel, fine sand	75	86
Medium to coarse gravel, medium sand	86	90
Clay balls	90	

observation wells; Mississippi River stages were available from the river gage at St. Louis.

A time-drawdown field data graph (figure 20) for well S-2 was superposed on the nonequilibrium type curve. The Theis (1935) equations were used to determine coefficients of transmissibility and storage of the aquifer for data on the third segment of the time-drawdown curve. The coefficient of transmissibility was computed to be 210,000 gpd/ft. The coefficient of storage (0.082) is in the water-table range. Drawdowns deviated from the type-curve trace during the latter part of the test because of the effects of induced infiltration. The distance to the image well associated with the recharge boundary was computed to be 1790 feet from the following equation (see Ingersoll, Zobel, and Ingersoll, 1948):

$$r_i = r_p\sqrt{t_i/t_p} \qquad (8)$$

where:

r_i = distance from image well to observation well, in ft

r_p = distance from pumped well to observation well, in ft

t_p = time after pumping started, before the boundary became effective, for a particular drawdown to be observed, in min

t_i = time after pumping started, after the boundary became effective, when the divergence of the time-drawdown curve from the type-curve trace under the influence of the image well is equal to the particular value of drawdown at t_p, in min

Specific-Capacity Data

The yield of a well may be expressed in terms of its specific capacity, which is defined as the yield in gallons per minute per foot of drawdown (gpm/ft) for a stated pumping period and rate. Walton (1962) gave an equation for computing the theoretical specific capacity of a well discharging at a constant rate in a homogeneous, isotropic, artesian aquifer infinite in areal extent.

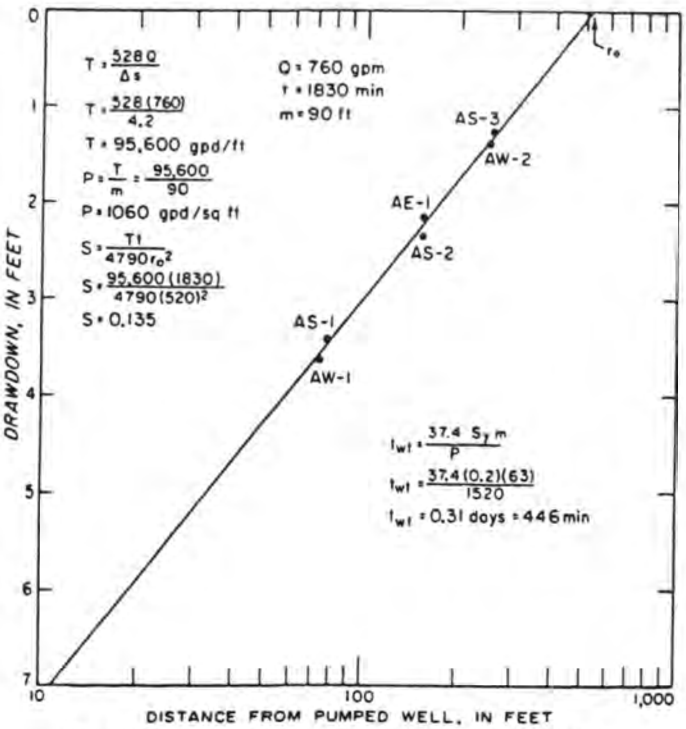


Figure 18. Distance-drawdown data for aquifer test 5

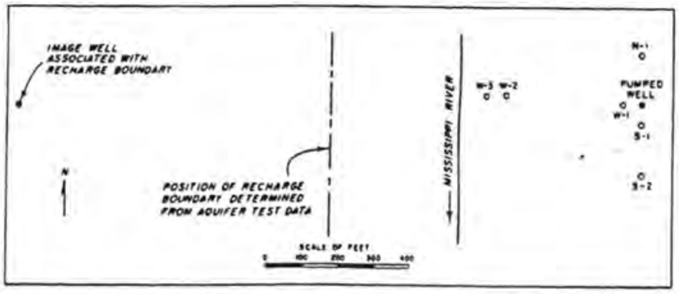


Figure 19. Location of wells used in aquifer test 6

Table 13. Drillers Logs of Wells Used in Aquifer Test 6

Formation	From (ft)	To (ft)
Well S-1		
Gray sandy clay	0	30
Gray fine sandy clay	30	40
Coarse gray sand, small gravel	40	45
Gray fine sand, scattered fine gravel, brown fine sand	45	66
Brown coarse sand, fine gravel	66	76
Coarse sand and gravel	76	90
Coarse sand, fine to medium gravel	90	100
Bedrock		About 120
Well S-2		
Gray sandy clay	0	30
Gray fine sandy clay	30	40
Coarse gray sand, small gravel	40	45
Gray fine sand, scattered fine gravel, brown fine sand	45	66
Brown coarse sand, fine gravel	66	75
Brown coarse sand, fine gravel, some gray clay	75	76
Coarse sand, small to large gravel	76	90
Brown coarse sand, fine to medium gravel	90	100

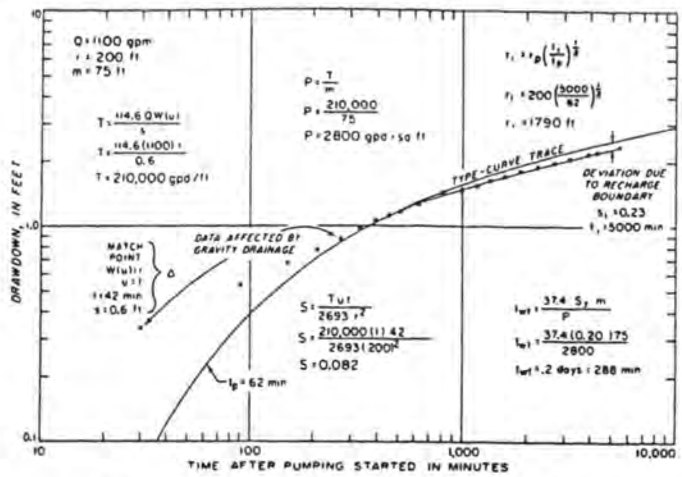


Figure 20. Time-drawdown data for well S-2, aquifer test 6

The specific capacity is influenced by the hydraulic properties of the aquifer, the radius of the well, r_w, and the pumping period, t. The relationship between the theoretical specific capacity of a well and the coefficient of transmissibility is shown in figure 21. A pumping period of 24 hours, a radius of 12 inches, and a storage coefficient of 0.1 were used in constructing the graph.

There is generally a head loss or drawdown (well loss) in a production well due to the turbulent flow of water as it enters the well itself and flows upward through the bore hole. Well loss and the well-loss coefficient may be computed by equations given by Jacob (1946). The computations for the well-loss coefficient, C, require data collected during a step-drawdown test

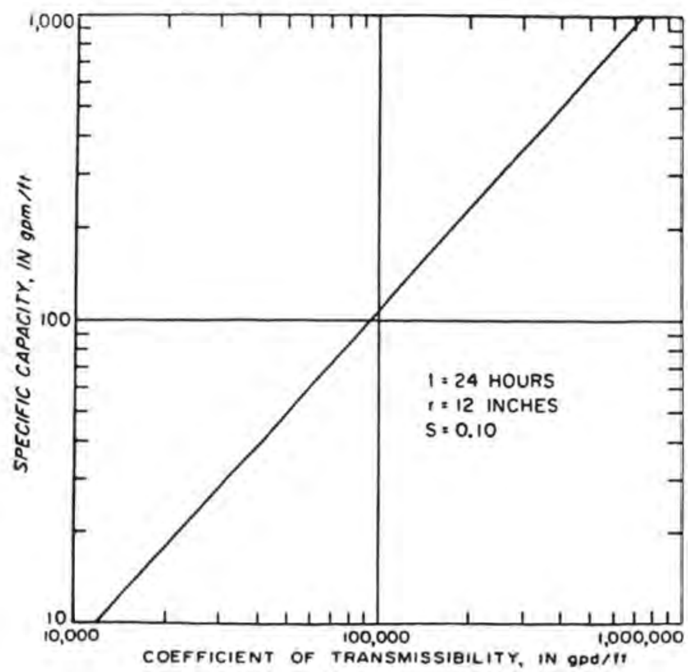


Figure 21. Theoretical relation between specific capacity and the coefficient of transmissibility

in which the well is operated during three successive and equal time periods at constant fractions of full capacity.

Step-drawdown test data are available for nine wells in the East St. Louis area. The results of the step-drawdown tests and construction features of the wells tested are given in table 14. Well-loss constants for wells tested immediately after construction range from 0.2 to 1.0 sec^2/ft^5.

Specific-capacity data collected during well-production tests made on 32 industrial, municipal, and irrigation wells are given in table 15. The well-production tests consisted of pumping a well at a constant rate and frequently measuring the drawdown in the pumped well. Drawdowns were commonly measured with an airline, electric dropline, or steel tape; rates of pumping were largely measured by means of a circular orifice at the end of the pump discharge pipe.

The lengths of tests ranged from 11 minutes to 2 days; pumping rates ranged from 104 to 1905 gpm. Screen diameters ranged from 8 to 32 inches.

Specific-capacity data for 65 selected relief wells are given in table 16. The wells were tested during the period 1952 through 1960 by the U.S. Corps of Engineers. The saturated thickness of the aquifer at well sites was estimated from logs of wells and water-level data. The tests consisted of pumping the wells at a constant rate of 500 gpm for 2 hours and frequently measuring the drawdown in the pumped well.

A coefficient of storage in the water-table range (0.10) estimated from aquifer-test data and several values of t and r_w were used (see Walton 1962) to determine the relationship between specific capacity and the coefficient of transmissibility for various values of r_w^2/t (figure 22). Specific capacities, data concerning the lengths of tests and radii of wells in tables 15 and 16, and figure 23 were used to estimate theoretical coefficients of transmissibility of the aquifer within the cones of depression of production wells. Theoretical coefficients of permeability within the cones of depression were estimated by dividing the coefficient of transmissibility by the average saturated thickness of the aquifer within cones of depression The average saturated thickness of the aquifer within cones of depression was estimated from logs of wells and water-level data. No great accuracy is implied for the coefficients of permeabilities estimated from specific-capacity data because they are based on an estimated coefficient of storage and are not corrected for well-loss and partial penetration losses. However, as shown in table 14, well-loss constants for most newly constructed wells are small. Most wells penetrate completely the more permeable parts of the aquifer. Thus, well and partial penetration losses were probably small and not significant. The data in tables 15 and 16 can be considered only rough approximations of the coefficient of permeability of the aquifer. However, the coefficients of permeability in the Monsanto area estimated from specific-capacity data agree closely with the coefficients of permeability computed from aquifer tests at the Mobil Oil Refinery and the Monsanto Chemical Corporation, indicating that the estimated coefficients of permeability are meaningful.

Water-level and pumpage data for existing pumping centers were used to compute pumping center specific capacities given in table 17. Pumping center specific capacity is here defined as the total pumpage from wells within the pumping center per foot of average drawdown within the pumping center.

Summary of Aquifer-Test Data

A map showing how the coefficient of permeability varies within the East St. Louis area (figure 23) was

Table 14. Results of Step-Drawdown Tests

Owner	Driller	Screen length (ft)	Screen diameter (in)	Screen Material	Date well drilled	Date of test	Well-loss constant (sec^2/ft^5)
Mobil Oil Co.	Luhr Bros.	36	16	Johnson Everdur No. 50 slot	12/59	10/61	2.0
Southwestern Campus of SIU, Edwardsville	Layne-Western	20	10	Slotted pipe	11/60	12/60	0.2
Collinsville (V)	Layne-Western	30	16	Layne No. 4 slot	8/50	8/50	0.7
Thomason	Thorpe		32 × 40	Porous concrete	5/54	5/54	0.2
Amos Bonham	Thorpe		30 × 40	Porous concrete	10/54	4/55	0.45
Herbert Bischoff	Thorpe	60	30 × 40	Porous concrete	1/54	5/54	0.5
V. W. Eckmann	Thorpe	48	30 × 40	Porous concrete	9/54	10/54	1.0
East St. Louis Drainage Dist.	Luhr Bros.			Wood	4/55	5/55	1.0
East St. Louis Drainage Dist.	Luhr Bros.			Wood	4/55	5/55	1.0

Table 15. Specific-Capacity Data for Industrial, Municipal, and Irrigation Wells

Well number	Owner	Depth (ft)	Diameter (in)	Screen length (ft)	Date of test	Length of test (min)	Nonpumping water level (ft below land surface)	Pumping rate (gpm)	Drawdown (ft)	Specific capacity (gpm/ft)	Coefficient of transmissibility (gpd/ft)	Saturated thickness of aquifer (ft)	Coefficient of permeability (gpd/sq ft)
MAD—5N10W-13.5c	Owens Illinois Glass Co.	68	26	20	3/40	390	9.3	1263	14.5	87	135,000	59	2300
5N9W-16.5b2	Olin Mathieson Chemical Corp.	90	26		11/33	60	46	560	9	62.2	62,000	44	1410
21.4h	Olin Mathieson Chemical Corp.	97	26		2/55	1440	56	300	11.7	25.6	42,000	41	1025
19.8h	Alton Boxboard Co.	110	26	35	1/57	480	30	1905	7.17	266	370,000	80	4630
22.1b1	Bethalto (V)	94	32	54	3/42	1440	41.5	320	5.2	62	80,000	53	1510
22.1b2	Bethalto (V)	94	32		3/42	600	41.5	305	4.25	72	100,000	53	1890
22.2c	Bethalto (V)	93	30	48	5/51	375	40	460	7	65	83,000	53	1570
26.8g1	City of Wood River	116	12	40	4/43	385	47.3	730	11	66.4	105,000	69	1520
26.8g2	City of Wood River	112	12	30	4/43	505	45.7	405	6	73.6	115,000	66	1740
26.8g3	City of Wood River	110	16	41	4/56	48	58	925	6	154	200,000	52	3850
26.8g4	City of Wood River	112	16	40	10/57	11	60.9	758	6.5	117	150,000	61	2460
27.1b	Roxana (V)	126	32	72	3/37	1440	48	530	6	88	120,000	78	1540
34.7c	International Shoe Co.	117	12	25	4/51	540	35	1125	17	61	94,000	82	1150
MAD—4N9W-13.1c1	Edwardsville (V)	112	16	41	2/40	45	25.5	1650	19	87	98,000	96	1020
29.7b	J. Thomason	106	30	60	4/54	100	24	1000	5.48	182	210,000	82	2560
3N8W-5.2f2	Glen Carbon (V)	63	30	48	5/56	150	30	104	7	14.9	19,000	33	575
29.3h1	Troy (V)	115	10	20	2/53	2880	25	420	6.35	66	120,000	90	1330
29.3h2	Troy (V)	115	12		11/60	60	25	325	3.1	105	110,000	90	1220
30.7b	V. W. Eckmann	104	30	48	10/54	40	20.3	468	5.38	87	180,000	64	2810
31.2a2	Collinsville (C)	104	26	30	8/58	480	18.8	1150	17.7	68	105,000	103	1020
31.2a3	Collinsville (C)	98	18	30	9/50	30	20.5	627	4.8	130	165,000	77	2150
31.2a4	Collinsville (C)	98	26	25	8/55	255	29.0	1001	11.0	91	130,000	69	1890
3N9W-5.8b	Herbert Bischoff	110	30	60	4/54	70	31.0	820	5.1	161	180,000	79	2280
6.3c	Herbert Bischoff	110	30	60	5/54	35	28.3	1120	7.88	140	140,000	82	1710
14.2c	W. Hanfelder	102	12	32	11/56	190	22.1	768	15.5	49.5	70,000	80	875
17.2a	Udell Bischoff	106	30	60	5/54	95	30.6	1150	5.78	199	230,000	75	3170
STC—2N8W-6.5h	Amos Bonham	106	30		4/55	60	28.4	470	6.55	72	90,000	67	1340
6.8d	E. A. Weissert	105	12	20	9/54	60	27	450	6	75	77,000	78	990
2N9W-1.3f	Mounds Public Water Dist.	90	8	10	7/58	900	8	349	10	34.9	60,000	82	730
2N10W-1.3a4	National Stockyards Co.	110	18	40	5/61	360	37	1248	8.18	152.5	200,000	73	2740
1.3a5	National Stockyards Co.	110	18	40	5/61	300	37	1230	6.55	188	250,000	73	3430
12.6g	Royal Packing Co.	100	12	40	1/59	475	33	475	3	158	190,000	67	2840

Table 16. Specific-Capacity Data for Selected Relief Wells

Well number	Date of test	Specific capacity (gpm/ft)	Coefficient of transmissibility (gpd/ft)	Estimated saturated thickness (ft)	Coefficient of permeability (gpd/sq ft)	Relief well number
Wood River (upper) Drainage District						
MAD—5N9W-						
13.2a	8/54	115	135,000	90	1500	41X
13.6d	8/54	238	305,000	60	5100	16
14.1e	9/54	62	67,000	60	1120	1
19.3c	1/55	96	110,000	96	1450	100
19.6e	1/55	156	190,000	96	1980	87XX
Wood River (lower) Drainage District						
MAD—5N9W-						
20.5a	10/60	114	134,000	80	1670	105
28.4c	10/60	101	118,000	100	1180	146
28.8e	10/60	89	100,000	100	1000	138
29.4g	10/60	65	72,000	100	720	121

Well number	Date of test	Specific capacity (gpm/ft)	Coefficient of transmissibility (gpd/ft)	Estimated saturated thickness (ft)	Coefficient of permeability (gpd/sq ft)	Relief well number
East St. Louis (Chain of Rocks) Drainage District						
MAD—4N9W-						
20.3g	8/52	94	108,000	85	1270	196
20.4e	6/52	66	72,000	90	800	184
20.5c	5/52	88	100,000	90	1110	175
20.6a	6/52	93	105,000	90	1160	170
29.7g	9/52	68	75,000	85	880	169
29.8d	9/52	79	88,000	80	1100	161
30.1b	9/52	66	72,000	75	960	155
31.2h	8/52	92	104,000	75	1390	150
31.3f	8/52	91	102,000	75	1360	144
31.3g	8/52	56	60,000	75	800	145
31.5c	7/52	91	102,000	70	1460	141
31.6a	7/52	77	86,000	70	1230	126

Table 16 (Continued)

Well number	Date of test	Specific capacity (gpm/ft)	Coefficient of transmissibility (gpd/ft)	Estimated saturated thickness (ft)	Coefficient of permeability (gpd/sq ft)	Relief well number
MAD—						
3N9W-						
6.7g	7/52	56	60,000	70	857	117
6.8e	8/52	91	103,000	70	1470	108
3N10W-						
1.1c	7/52	22	21,000	70	300	98
12.4f	6/52	103	120,000	70	1720	69
12.6c	6/52	49	52,000	70	743	56
13.8g	4/52	58	62,000	50	1240	38
14.1f	9/52	38	39,000	50	780	33
14.2d	4/52	31	31,000	50	620	26
23.5g	9/52	44	46,000	45	1020	5
East St. Louis Drainage District						
MAD—						
4N8W-						
7.3a	11/58	172	212,000	80	2660	1
4N9W-						
14.8h	10/58	61	66,000	85	780	3
3N10W-						
22.1a	7/55	15	14,000	30	456	43
23.6c	6/55	41	43,000	40	1070	7
26.6b	7/55	33	33,000	40	825	78
26.7d	7/55	25	24,000	40	600	70
26.8e	7/55	64	70,000	35	2000	64
26.8h	7/55	34	35,000	30	1170	53
35.6f	7/55	39	40,000	45	890	96
35.6h	7/55	36	37,000	45	823	87
STC—						
2N10W-						
11.4e	10/54	131	156,000	75	2080	131
14.4h	10/54	95	110,000	85	1290	107
23.5h	8/55	156	190,000	90	2110	124
23.6c	8/55	143	175,000	85	2060	129
23.6f	7/55	143	175,000	80	2190	118
23.7a	8/55	139	165,000	80	2060	136
34.5h	8/55	236	300,000	90	3340	159
34.6e	10/54	109	125,000	95	1320	159
34.7c	10/54	151	182,000	95	1910	169
1N10W-						
4.1g	10/54	89	100,000	95	1050	196
4.2e	9/54	113	134,000	90	1490	207
4.3b	11/54	142	175,000	85	2060	237
9.1f	10/58	66	72,000	100	720	262
9.2h	10/58	116	136,000	95	1430	251
10.1c	10/58	125	148,000	80	1850	273
10.4c	10/58	104	120,000	80	1500	263
12.5b	10/58	132	160,000	65	2460	278
13.3h	10/58	126	150,000	60	2500	286
Prairie Du Pont Drainage District						
STC—						
1N10W-						
4.7b	10/54	126	150,000	70	2140	23
8.2h	10/54	148	180,000	55	3280	28
8.5c	10/54	84	96,000	80	1200	34
8.7a	10/54	103	120,000	65	1850	45
9.4h	10/54	125	150,000	70	2140	15
19.6f	11/54	91	103,000	60	1720	46
30.6h	10/54	130	154,000	85	1810	55

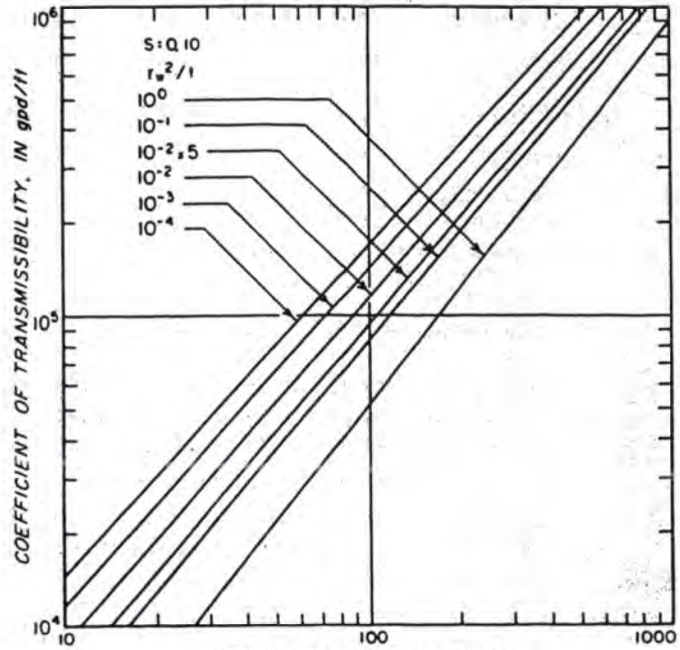


Figure 22. Coefficient of transmissibility versus specific capacity for several values of well radius and pumping period

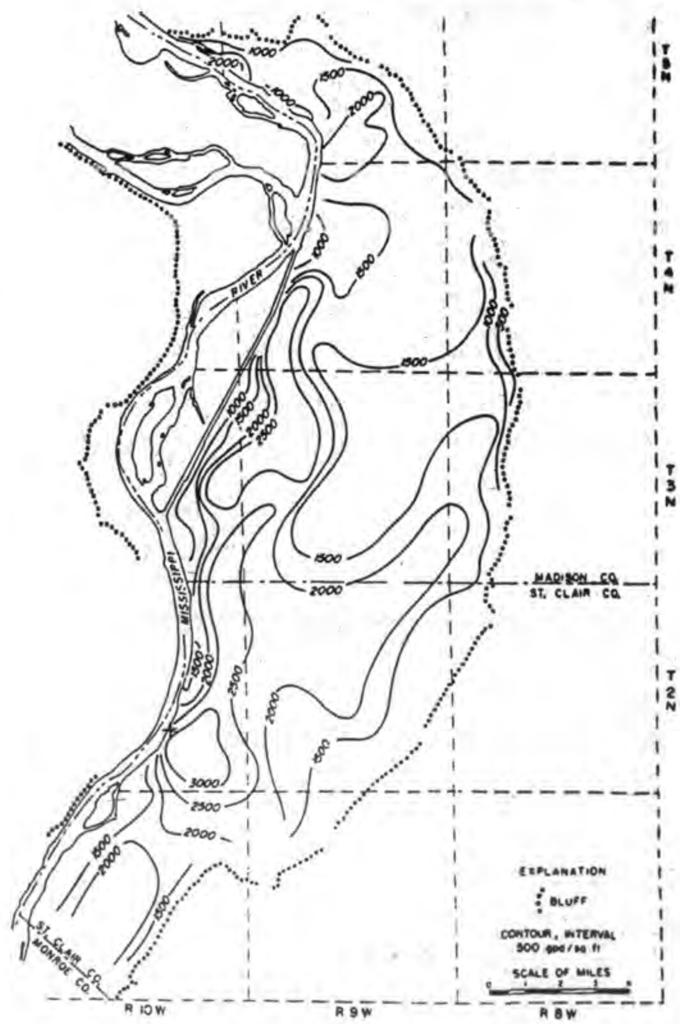


Figure 23. Coefficient of permeability of aquifer

5.56

Table 17. Pumping Center Specific-Capacity Data

Pumping center	Pumpage in 1961 (mgd)	Average drawdown (ft)	Specific capacity (gpd/ft)
Alton	5.1	20	255,000
Wood River	13.5	40	338,000
Granite City	8.8	15	586,000
National City	10.8	20	540,000
Monsanto	20.5	50	410,000

prepared from data in tables 14, 15, and 16. The coefficient of permeability is high in narrow strips extending from Monsanto north through National City and extending through Granite City northeasterly along the Chain of Rocks Canal. The coefficient of permeability is greatest locally in the Monsanto area, exceeding 3000 gpd/sq ft. The coefficient of permeability is estimated to be greater than 2000 gpd/sq ft south of Alton (along the Mississippi River) in the Wood River area, in a wide area extending from Monsanto northeast to just south of Horseshoe Lake, and in the Dupo area. The coefficient of permeability is less than 1000 gpd/sq ft in an area extending south from near the confluence of the Missouri and Mississippi Rivers to north of Horseshoe Lake. The coefficient of permeability decreases rapidly near the bluffs and west of the Chain of Rocks Canal.

A map showing the saturated thickness of the aquifer (figure 24) was prepared from the bedrock surface map (figure 6), water-level data for November 1961, and a map showing the elevation of the base of the alluvium. The saturated thickness of the aquifer is greatest and exceeds 100 feet in the bedrock valley bisecting the East St. Louis area. It is least along the bluffs and west of Chain of Rocks Canal.

A map showing how the coefficient of transmissibility varies within the East St. Louis area (figure 25) was prepared from figures 23 and 24. The coefficient of transmissibility ranges from less than 50,000 gpd/ft near the bluff and the southern part of the Chain of Rocks Canal to greater than 300,000 gpd/ft near Monsanto.

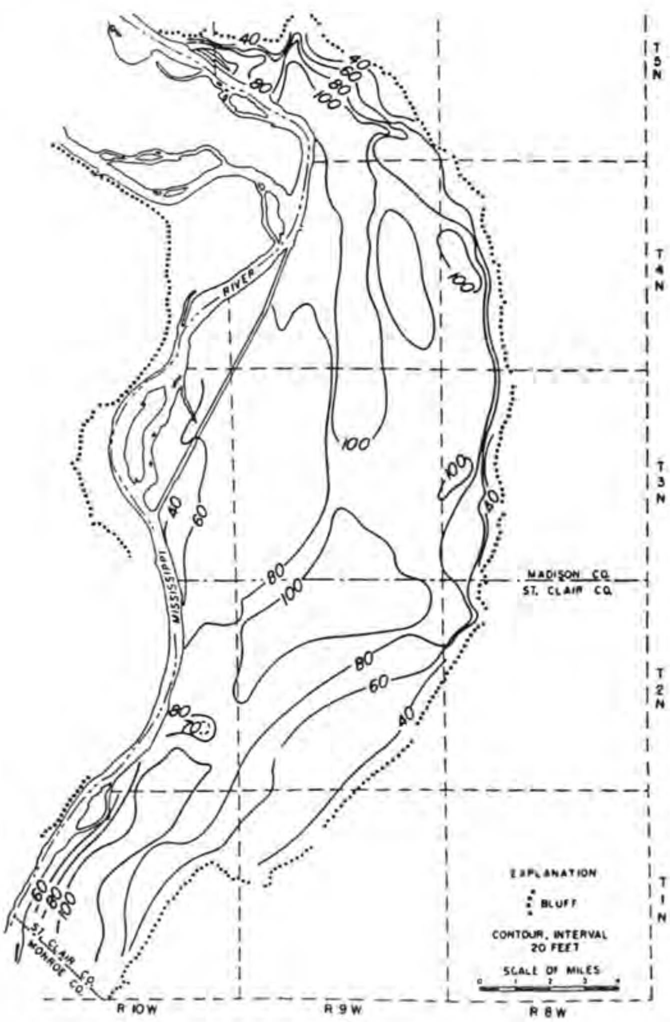


Figure 24. Saturated thickness of aquifer, November 1961

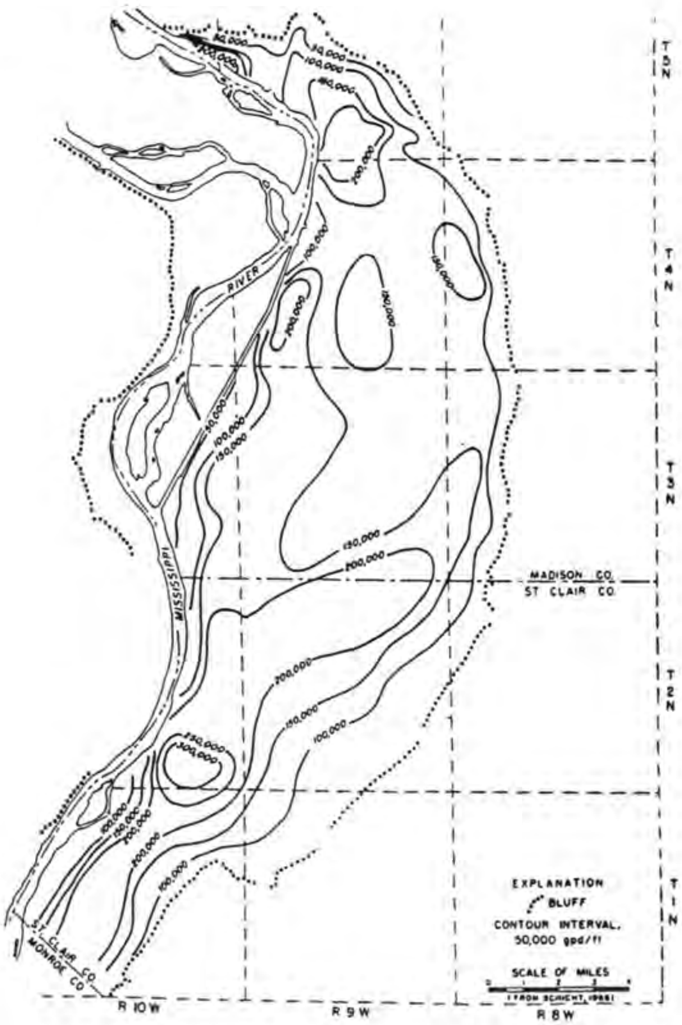


Figure 25. Coefficient of transmissibility of aquifer

CONSTRUCTION FEATURES AND YIELDS OF WELLS

Large capacity wells in the East St. Louis area are drilled by the cable tool method, the reverse hydraulic rotary method, or by clam shell type diggers. Collector wells have been constructed in the East St. Louis area by several industries. Most domestic and some irrigation wells are driven; a few dug wells are still used for domestic supplies.

Industrial, municipal, and irrigation wells are usually drilled to bedrock or bit refusal. Several wells just south of Alton terminate at the top of clayey and silty material immediately above bedrock. According to Bergstrom and Walker (1956) the maximum thickness of the clayey and silty material is 25 feet. Production wells are usually cased through the finer alluvial deposits in the upper part of the valley fill and have perforated pipe sections or commercial screens opposite the lower coarser alluvium or valley-train deposits. There are two types of drilled wells in the area: natural pack and artificial pack. Materials surrounding the well are developed in place in the case of the natural pack well; materials having a coarser and more uniform grain size than the natural formation are added around the well in the case of the artificial pack well. As shown in table 18, the thickness of the pack in wells in the area generally ranges from 6 to 11 inches.

Table 18. Construction Features of Selected Wells

			Screen Record				
Depth (ft)	Casing depth (ft)	Casing diameter (in)	Length (ft)	Diameter (in)	Material or manufacturer	Slot number or size (in)	Artificial pack thickness (in)
103	0-73	26	30	26	Everdur Johnson	30	11
110	0-34	26 36	76	26 36	Porous concrete		none
85	0-49	30 40	36	30 40	Porous concrete		none
95	0-47	26 36	48	26 36	Porous concrete		none
120	0-76	16	44	16	Slotted pipe	¼ × 2½	9.5
108	0-73	24	35	24	Everdur Johnson	60 100	6.0
100	0-63	14	37	12	Slotted pipe	¼ × 2½	7.0
105	0-85	12	20	12	Slotted pipe		6.0
111	0-81	16	30	16	Cook	20 30	
114	0-84	16	32	16	Cook	20 40	
111	0-81	16	30	16	Cook	20 40 80	
98	0-78	18	20	18	Layne Shutter	4	
115	0-85	16	30	16	Cook	30	6.0
105	0-89	10	16	10	Cook		none
115	0-100	12	15	12	Johnson	60	none

Several types of well screens have been used in the East St. Louis area. Porous concrete, wood, slotted pipe, and commercial screens are in use. Economic considerations rather than proper well design criteria have governed the types of screens in use. Screen diameters generally vary in diameter from 6 to 30 inches, and screens vary in length from 5 to 76 feet. Screen slot openings vary depending upon the characteristics of the formations encountered or the characteristics of the artificial pack.

Ten collector wells have been constructed in the East St. Louis area, and six are still in use. Four collector wells at the Granite City Steel Company were not in continuous operation in 1962, but were tested periodically and operated occasionally during the summer months. The collector well consists of a large diameter, reinforced concrete caisson from which horizontal screen laterals project radially near the bottom. The standard caisson is 13 feet in diameter. The horizontal screen laterals are fabricated from heavy steel plate, perforated with longitudinal slots, and may be 8 to 24 inches in diameter and 100 to 450 feet in length, depending upon geologic conditions and design of the unit (Mikels and Klaer, 1956).

Thorpe concrete wells are in wide use by municipalities, industries, and irrigation well owners. Thorpe concrete wells consist of a concrete casing and porous concrete screen either 26 or 30 inches in inside diameter with walls 5 inches thick. Lengths of screen vary from 24 to 76 feet. Thorpe concrete wells have been in operation for as long as 35 years. However, in some cases Thorpe concrete wells have been abandoned because of reduction in yield after a few months operation.

Driven wells are usually not greater than 50 feet in depth depending upon the thickness of the alluvium overlying the coarser sand and gravel deposits. The driven wells consist of lengths of 1.25- or 2-inch diameter pipe with a drive (or sand) point at the lower end of the pipe.

About 500 relief wells were drilled in the East St. Louis area by the U.S. Corps of Engineers near and on the land side of levees fronting the Mississippi River to control underseepage beneath levees during floods. Several artificial pack relief wells were also drilled along the Cahokia Diversion Channel. Relief wells in the area range in depth from 47 to 103 feet. Casings and screens are 8 inches in diameter and the pack thickness is about 7 inches. The screens are constructed from redwood or treated Douglas Fir and range in length from 19 to 71 feet. The screens are spiral wound with No. 6 gage galvanized wire and have 18 slots, 3/16 by 3 1/4 inches per spiral.

Slotted pipe screens are widely used in irrigation wells in the East St. Louis area because of their low cost. In comparison, only a few industrial and municipal ells contain slotted pipe screens. Irrigation wells range n diameter from 8 to 16 inches and usually have pack thicknesses of 6 to 8 inches. Lengths of slotted pipe screens range from 10 to 40 feet.

Service Life of Wells and Collector Wells

One of the problems in the East St. Louis area associated with the development of ground-water resources is the short life expectancy of wells. According to a study by Bruin and Smith (1953), the median service life of municipal wells terminating in sand and gravel formations in the East St. Louis area is about half that for similar municipal wells in other parts of the state. Nearly all of the wells retired in the area were taken out of service either because the screens had become partially clogged or the wells had filled with sand.

The results of mechanical analyses presented by Bergstrom and Walker (1956) are shown in figures 26 through 28. According to Bergstrom and Walker the analyses must be accepted with caution because the conditions of collecting most of the samples are not known, and because of the highly variable nature of the valley-fill deposits in the area. A careful examination of the mechanical analysis curves suggests that the valley-fill deposits contain a rather high percentage of fine materials which could, under heavy pumping conditions, migrate toward a screen and partially clog the well wall and screen openings. As indicated by data in the files of industries and municipalities, specific capacities of existing production wells decrease markedly after a few years and in some cases after a few months of operation. Specific capacities are generally determined by the driller after completion of the well by pumping the well at different rates for short periods of time, generally less than 24 hours, and by frequently measuring drawdowns in the pumped well. This method of measuring specific capacity is continued by industrial and municipal personnel periodically.

It is a general practice of industries and municipalities to place a well in operation and pump it at high rates, often about 1000 gpm. As the result of heavy pumping, fine materials migrate towards the well and partially clog screen openings and the voids of the formation surrounding the well. The well-loss constant increases rapidly and, because well loss varies as the square of the discharge rate, drawdown increases rapidly. The relation between well-loss constant and drawdown due to well loss is shown in figure 29. As drawdown increases the specific capacity and, therefore, the yield of the well decreases. Typical decreases in specific capacity due to increases in the well-loss constant are given in table 19.

Theoretical specific capacities of wells with a nominal radius of 15 inches and with 40 feet of screen given in table 19 were determined for values of the coefficient of transmissibility ranging from 100,000 to 300,000 gpd/ft,

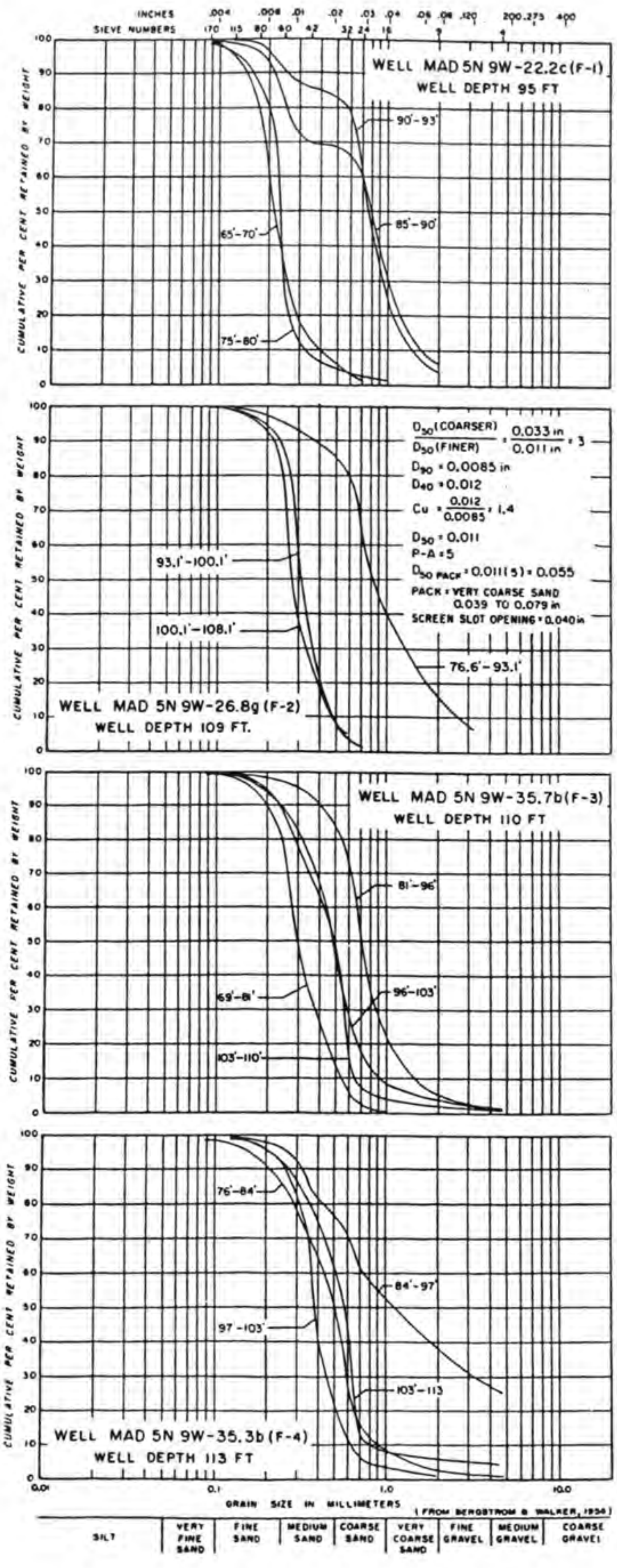


Figure 26. Mechanical analyses of samples from wells

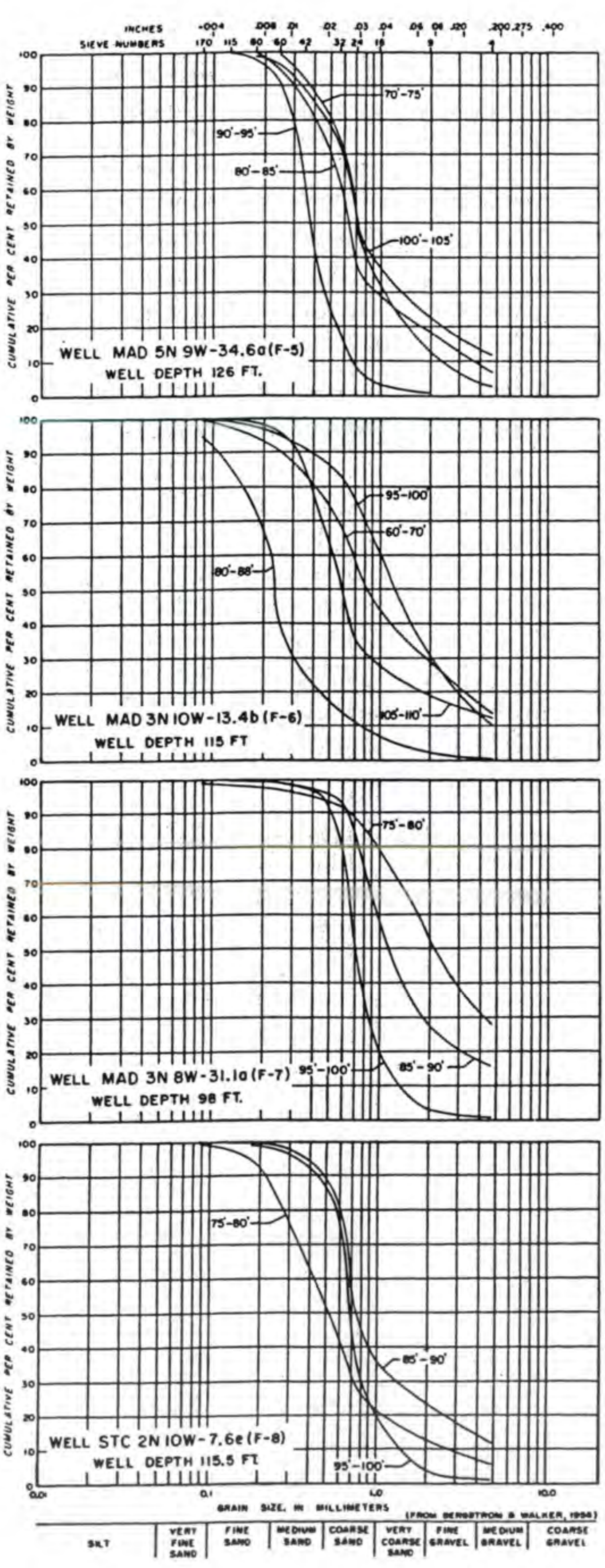


Figure 27. Mechanical analyses of samples from wells

a coefficient of storage of 0.10, a pumping period of 12 hours, pumping rates of 900 or 450 gpm, well-loss constants of 1, 5, and 10 sec^2/ft^5. The effects of dewatering and partial penetration (see Walton, 1962) were taken into consideration in computations.

Computed well-loss coefficients for wells tested immediately after construction (table 14) range from 0.2 sec^2/ft^5 to 1.0 sec^2/ft^5 and meet requirements suggested by Walton (1962) that the value of C of a properly developed and designed well should be less than 5 sec^2/ft^5. According to Walton (1962), values of C between 5 and 10 sec^2/ft^5 indicate mild deterioration, and clogging is severe when C is greater than 10 sec^2/ft^5. It is difficult

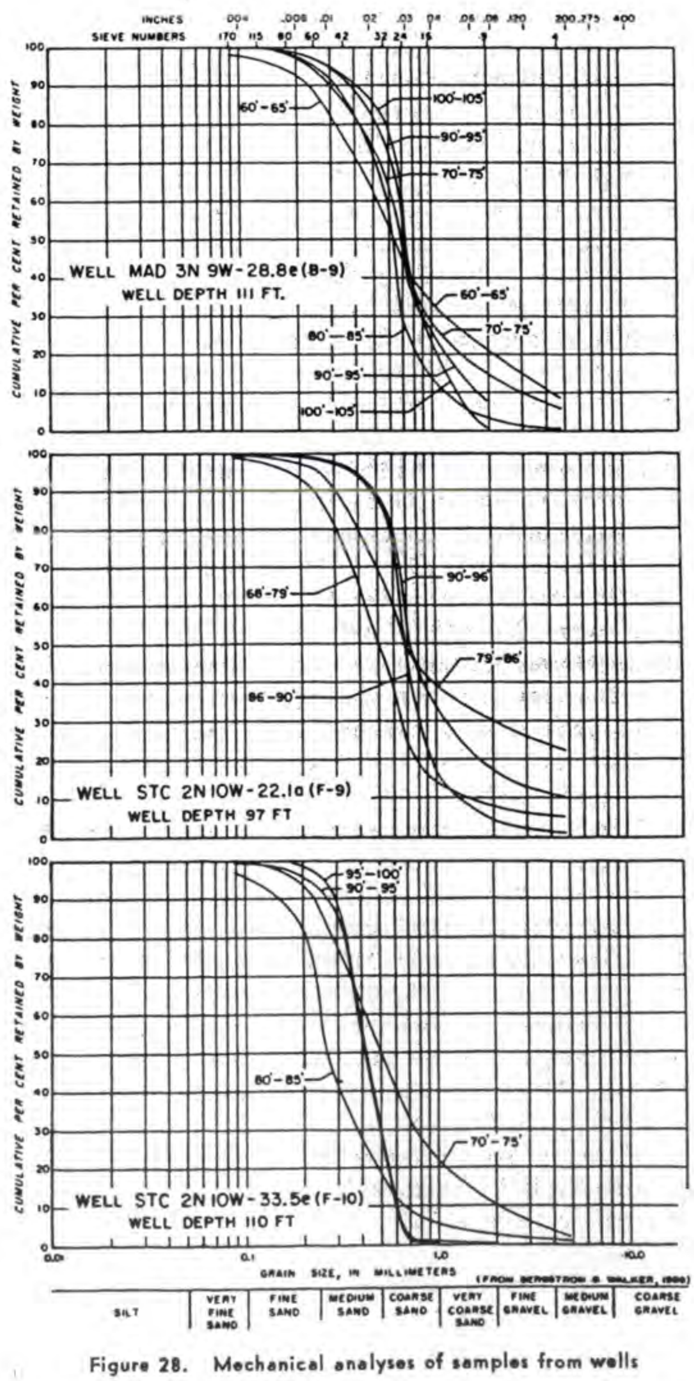


Figure 28. Mechanical analyses of samples from wells

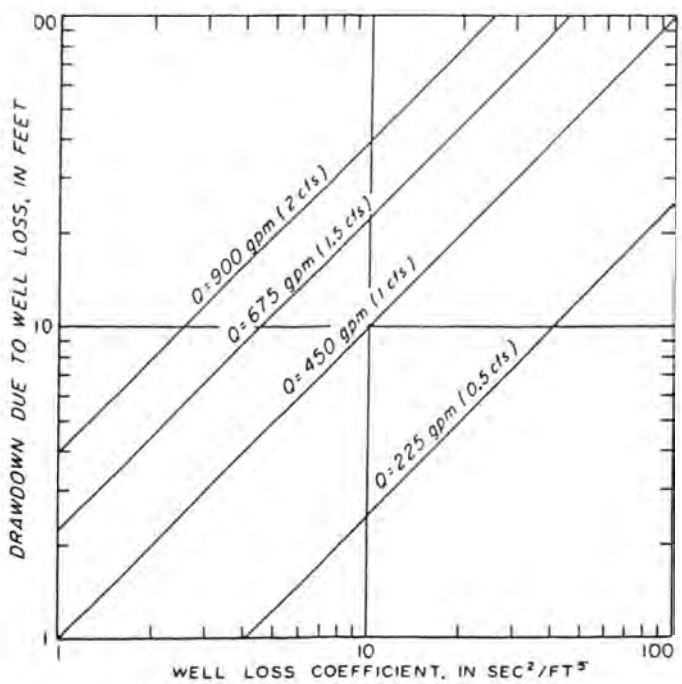


Figure 29. Relation between well-loss constant and drawdown due to well loss

and sometimes impossible to restore the original capacity if the well-loss constant is greater than 40 sec²/ft⁵.

Periodic well treatment by acidizing or other methods has been used successfully to rehabilitate old wells. However, in many cases wells are abandoned as their yields decrease and new wells are drilled nearby.

Based on data for production wells which have been in service a number of years, the average specific capacity of wells in the East St. Louis area is about 30 gpm/ft. An average well yield of 450 gpm can be obtained with a long service life if sufficient screen is provided.

A graph showing the decrease of specific capacity of a collector well owned by the Shell Oil Refinery near the city of Wood River is given in figure 30. The specific capacity of the collector well declined from a peak of 270 gpm/ft in August 1954 to about 50 gpm/ft in March 1963. A part of the decline in specific capacity can be attributed to the partial clogging of the laterals by incrustation and with sand and silt. Mechanical cleaning of one lateral in June 1962 increased the specific capacity from about 50 gpm/ft to 55 gpm/ft.

Figure 30. Specific-capacity data for collector well, 1954 to March 1963

Table 19. Theoretical Decreases in Specific Capacity Due to Increases in Well-Loss Constant

Coefficient of transmissibility (gpd/ft)	Pumping rate (gpm)	Well-loss coefficient of 1 sec²/ft⁵: Drawdown* (ft)	Well-loss coefficient of 1 sec²/ft⁵: Specific capacity* (gpm/ft)	Well-loss coefficient of 5 sec²/ft⁵: Drawdown* (ft)	Well-loss coefficient of 5 sec²/ft⁵: Specific capacity* (gpm/ft)	Well-loss coefficient of 10 sec²/ft⁵: Drawdown* (ft)	Well-loss coefficient of 10 sec²/ft⁵: Specific capacity* (gpm/ft)
300,000	900	9.3	96.9	25.3	35.6	45.3	19.9
250,000	900	10.3	87.4	26.3	34.2	46.3	19.4
200,000	900	11.9	75.6	27.9	32.2	47.9	18.8
150,000	900	14.4	62.5	30.4	28.6	50.4	17.9
100,000	900	19.7	45.7	35.7	25.2	55.7	16.1
300,000	450	3.7	122.2	7.7	58.4	12.7	35.4
250,000	450	4.2	110.7	8.2	54.9	13.2	34.1
200,000	450	4.9	91.9	8.9	50.6	13.9	32.4
150,000	450	6.1	73.8	10.1	44.5	15.1	29.8
100,000	450	8.4	53.6	12.4	36.3	17.4	25.9

*Theoretical

Well Design Criteria

Walton (1962) gave criteria for well design in unconsolidated formations in Illinois. Screen design criteria are applicable to industrial, municipal, and irrigation wells. The objective is to design an efficient and economical well with a service life of at least 10 years.

According to Ahrens (1957) artificial pack wells are usually justified when the aquifer is homogeneous, has a uniformity coefficient less than 3.0, and/or has an effective grain size less than 0.01 inch. The uniformity coefficient, C_u, is the ratio of the sieve size that will retain 40 percent of the aquifer materials to the effective size. The sieve size that retains 90 percent of the aquifer materials is the effective size. In addition, an artificial pack is sometimes needed to stabilize well-graded aquifers having a large percentage of fines in order to avoid excessive settlement of materials above the screen or to permit the use of larger screen slots. The uniformity coefficients based on mechanical analyses of samples in figures 26 through 28 are less than 3 and/or the effective grain size is less than 0.01 inch, indicating that an artificial pack well should be constructed at each site.

Selection of the artificial pack is based on the mechanical analysis of the aquifer. A criterion that has been successfully used in Illinois is that the ratio of the 50 percent sizes of the pack and the aquifer (the P-A ratio) be 5 (Smith, 1954). Artificial packs should range in thickness from 6 to 9 inches (Walton, 1962).

To avoid segregation or bridging during placement, a uniform grain size pack should be used. The screen slot opening should be designed so that at least 90 percent of the size fractions of the artificial pack are retained.

A well sometimes encounters several layers of sand and gravel having different grain sizes and gradations. If the 50 percent size of the materials in the coarsest aquifer are less than 4 times the 50 percent size of the materials in the finest aquifer, the slot size and pack, if needed, should be selected on the basis of the mechanical analysis of the finest material (Ahrens, 1957). Otherwise, the slot size and pack should be tailored to individual layers.

One of the most important factors in the design of natural pack well screens is the width or diameter of the screen openings, referred to as slot size. With a uniformity coefficient greater than 6 (a heterogeneous aquifer) and in the case where the materials overlying the aquifer are fairly firm and will not easily cave, the sieve size that retains 30 percent of the aquifer materials is generally selected as the slot size. With a uniformity coefficient greater than 6 and in the case where the materials cave, the sieve size that retains 50 percent of the aquifer materials is selected as the slot size (Walton, 1962). With a uniformity coefficient as low as 3 (a homogeneous aquifer) and in the case where the materials overlying the aquifer are fairly firm and will not easily cave, the sieve size that retains 40 percent of the aquifer materials is selected as the slot size. With a uniformity coefficient as low as 3 and in the case where the materials overlying the aquifer are soft and will easily cave, the sieve size that retains 60 percent of the aquifer materials is selected as the slot size.

The screen length is based in part on the effective open area of a screen and an optimum screen entrance velocity. According to Walton (1962), to insure a long service life by avoiding migration of fine materials toward the screen and clogging of the well wall and screen openings, screen length is based on velocities between 2 and 12 feet per minute (fpm).

The length of screen for a natural pack well is selected from the coefficient of permeability of the aquifer determined from aquifer tests by using table 20 and the following equation (Walton, 1962):

$$L_s = Q/A_e V_c (7.48) \qquad (9)$$

where:

L_s = required length of screen, in ft
Q = discharge, in gpm
A_e = effective open area per foot of screen, in sq ft
V_c = optimum entrance velocity, in fpm

On the average about one-half the open area of the screen will be blocked by aquifer materials. Thus, the effective open area averages about 50 percent of the actual open area of the screen.

Table 20. Optimum Screen Entrance Velocities*

Coefficient of permeability (*gpd/sq ft*)	Optimum screen entrance velocities (*fpm*)
>6000	12
6000	11
5000	10
4000	9
3000	8
2500	7
2000	6
1500	5
1000	4
500	3
< 500	2

From Walton (1962)

The results of studies involving the mechanical analyses of samples of the aquifer collected at two sites demonstrate some of the principles involved in the design of sand and gravel wells. Suppose that it is desired to design a 16-inch diameter well based on the mechanical analysis of samples for well MAD 5N9W-26.8g (see figure 26). Since the ratio of the 50 percent grain size of the coarser material from 76.6 to 93.1 feet to the 50 percent grain size of the finer material from 93.1 to 108.1 feet is less than 4, the screen or pack must be designed on the basis of results of analysis of the finer materials. The uniformity coefficient of the finer materials is less than 3 and the effective grain size is less than 0.01 inches, indicating that an artificial pack well should be used. The 50 percent size of the materials of the finest sample is 0.011 inch; thus, with a pack-aquifer ratio of 5, a very coarse sand pack with particles ranging in diameter from about 0.04 to 0.08 inch is indicated. To retain 90 percent of the size fractions of the pack a slot size of 0.040 inch would be required. An artificial pack thickness of 6 inches is adequate.

For demonstration of the design of a natural pack well, consider the grain-size distribution curves in figure 31. The mechanical analyses are for samples taken from

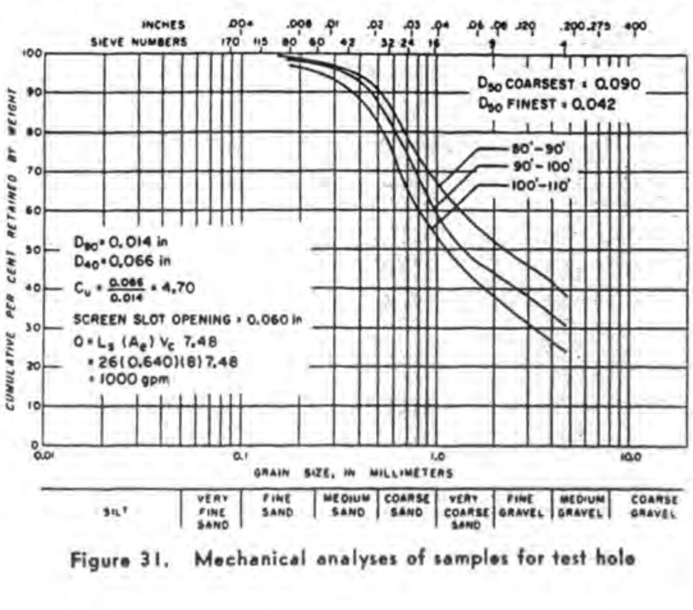


Figure 31. Mechanical analyses of samples for test hole

test hole near Monsanto. The coefficient of permeability the aquifer in the vicinity of the test hole was estimated to be 3000 gpd/sq ft from aquifer-test data. The 50 percent size of the materials in the finest sample is less than 4 times the 50 percent size of the materials in the coarsest sample; therefore, the slot size should not be tailored to individual samples but should be based on the mechanical analysis of the finest sample. The effective grain sizes of all three samples are greater than 0.01 and uniformity coefficients are greater than 3. A natural pack well is therefore indicated. The materials overlying the aquifer will not easily cave so the sieve size (0.060 inch) that retains 40 percent of the aquifer materials is selected as the proper slot size.

Suppose a pumping rate of 1000 gpm is desired. Computations made with equation 9, indicate that 26 feet of 16-inch continuous slot screen with a slot opening of 0.060 inches is needed. The effective open area of the screen is estimated to be 0.640 sq ft per foot of the screen. The optimum screen entrance velocity (table 20) is equal to 8 fpm.

Alternate designs to the above example are possible by using a small diameter screen with a longer length or a larger diameter screen with a shorter length.

The following are well diameters that have been used in Illinois (Smith, 1961):

Pumping rate *(gpm)*	Diameter of well *(in)*
125	6
300	8
600	10
1200	12
2000	14
3000	16

Experience has shown that in the case of a multiple well system consisting of more than two wells the proper spacing between wells is at least 250 feet.

GROUND-WATER WITHDRAWALS

The first significant withdrawal of ground water in the East St. Louis area started in the late 1890s. Prior to 1900 ground water was primarily used for domestic and farm supplies; since 1900 pumpage has been mostly for industrial use. The first record of an industrial well in the East St. Louis area is for a well drilled in 1894 by the Big Four Railroad in East Alton (Bowman and Reeds, 1907). The well was 54 feet deep and 8 inches in diameter, and was pumped at an average rate of 75,000 gpd. The water was used primarily in locomotive boilers. The meat packing industry in National City started to pump large quantities of ground water in 1900. According to Schicht and Jones (1962), estimated pumpage from wells in the National City area increased from 400,000 gpd in 1900 to 5.3 mgd in 1910. The first municipal well was drilled in 1899 by Edwardsville at a site near Poag and was pumped at an average rate of 300,000 gpd. The second municipal well was drilled in 1901 by Collinsville at a site about a mile north of Caseyville and was pumped at an average rate of 100,000 gpd. Pumpage from wells in the East St. Louis area from 1890 through 1960 was estimated by Schicht and Jones (1962). Estimated pumpage from wells increased from 2.1 mgd in 1900 to 111.0 mgd in 1956 as shown in figure 32. Pumpage declined sharply from 111.0 mgd in 1956 to 92.0 mgd in 1958 and then gradually increased to 93.0 mgd in 1960. The average rate of pumpage increase for the period 1890 through 1960 was about 1.5 mgd per year.

Pumpage from wells in the East St. Louis area was greatest in 1956, totaling 111.0 mgd. As shown in figure 32 pumpage increased from 93.0 mgd in 1960 to 96.8 mgd in 1961, and increased sharply to 105.0 mgd in 1962.

Pumpage is concentrated in five major pumping centers: the Alton, Wood River, Granite City, National City, and Monsanto areas. Also, there are five minor pumping centers: the Fairmont City, Caseyville, Poag, Troy, and Glen Carbon areas. The distribution of pumpage in 1956 and 1962 are shown in figures 33 and 34 respectively, which also indicate the locations of the pumping centers. As shown in figures 35 and 36, changes in pumpage for the period of record are similar in all major pumping centers. Poor economic conditions are reflected in the decreased pumpage during the years of the late 1920s and early 1930s. The effects of increased production dur-

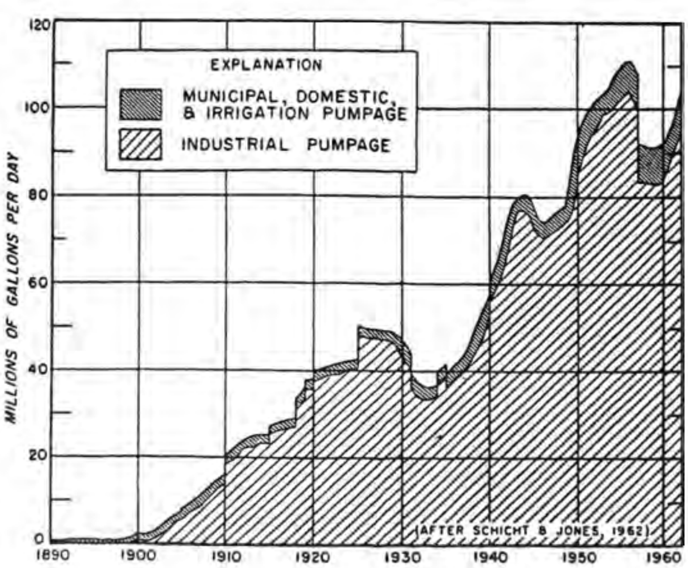


Figure 32. Estimated pumpage from wells, 1890 through 1962, subdivided by use

ing World War II and the post-war reduction in production are evident. There has been a general and gradual increase in pumpage from the five minor pumping centers throughout the period of record as shown in figure 37.

The distribution of pumpage from wells in 1956, 1960, 1961, and 1962 is shown in table 21. The greatest

Table 21. Distribution of Pumpage from Wells

Pumping center	Total pumpage (*mgd*) 1956	1960	1961	1962
Alton area	9.8	13.6	12.3	13.9
Wood River area	21.1	20.9	24.3	25.5
Granite City area	30.1	7.9	8.8	9.5
National City area	13.8	9.6	10.8	11.6
Monsanto area	30.1	33.2	31.9	35.4
Fairmont City area	2.4	3.2	4.4	4.5
Caseyville area	2.3	2.6	2.4	2.5
Poag area	0.9	1.2	1.2	1.2
Troy area	0.3	0.5	0.4	0.5
Glen Carbon area	0.2	0.3	0.3	0.4
Total	111.0	93.0	96.8	105.0

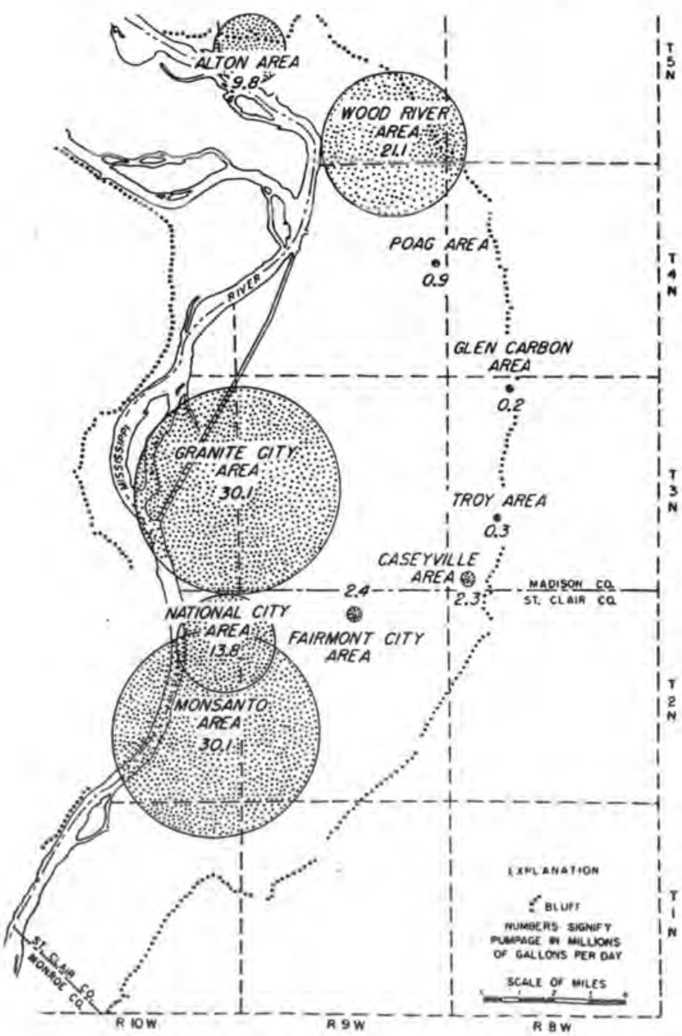


Figure 33. Distribution of estimated pumpage in 1956

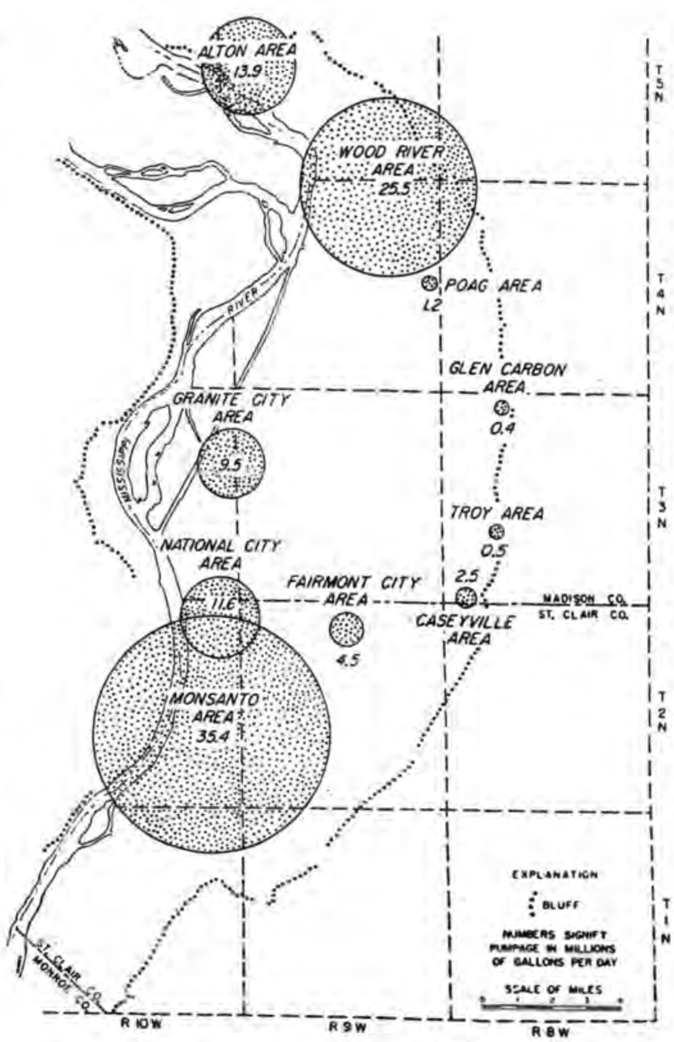


Figure 34. Distribution of estimated pumpage in 1962

change in pumpage from 1956 to 1962 occurred in the Granite City area. Because of a serious decline in water levels caused by heavy pumpage concentrated in a relatively small area and the severe drought during 1952-1956, the Granite City Steel Company abandoned its wells in 1957 and began obtaining water supplies from the Mississippi River. As a result, withdrawals of ground water dropped sharply from 30.1 mgd in 1956 to 7.6 mgd in 1958, and gradually increased to 9.5 mgd in 1962. Pumpage in the National City area in 1962 does not include pumpage necessary to dewater a cut along an interstate highway in construction near National City since this information was not available at the time this report was written.

Of the 1962 total pumpage, withdrawals for public water-supply systems amounted to about 6.4 percent, or 6.7 mgd; industrial pumpage was about 91.1 percent, or 95.7 mgd; domestic pumpage was 2.3 percent, or 2 mgd; and irrigation pumpage was 0.2 percent, or 0.2 mgd.

The major industries in the East St. Louis area using ground water are oil refineries, chemical plants, ore re-

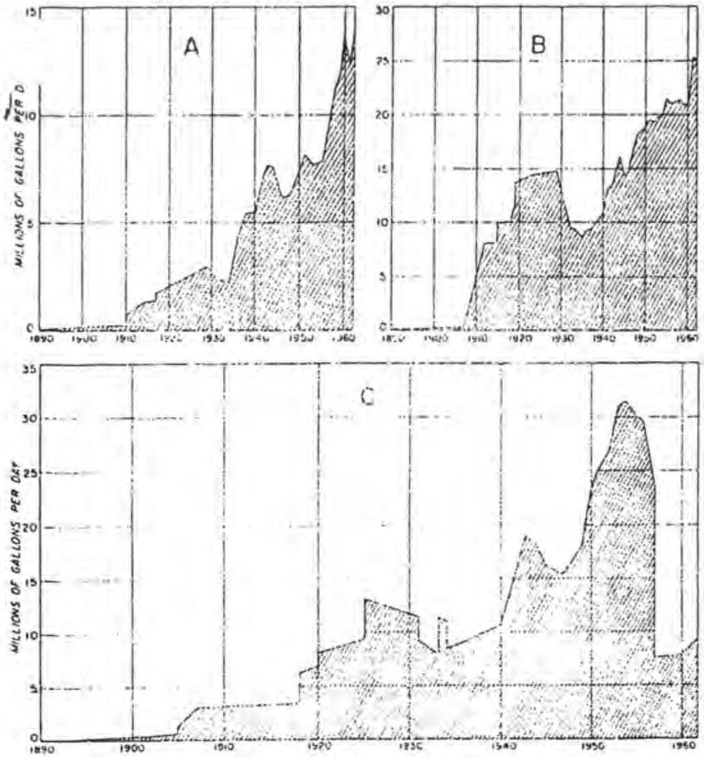


Figure 35. Estimated pumpage, Alton area (A), Wood River area (B), and Granite City area (C), 1890-1962

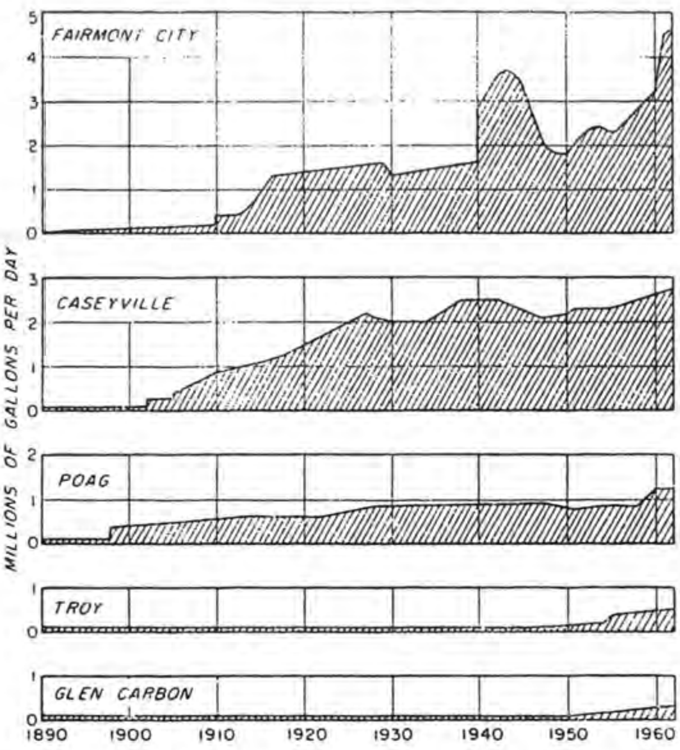


Figure 37. Estimated pumpage, Fairmont City, Caseyville, Poag, Troy, and Glen Carbon, 1890-1962

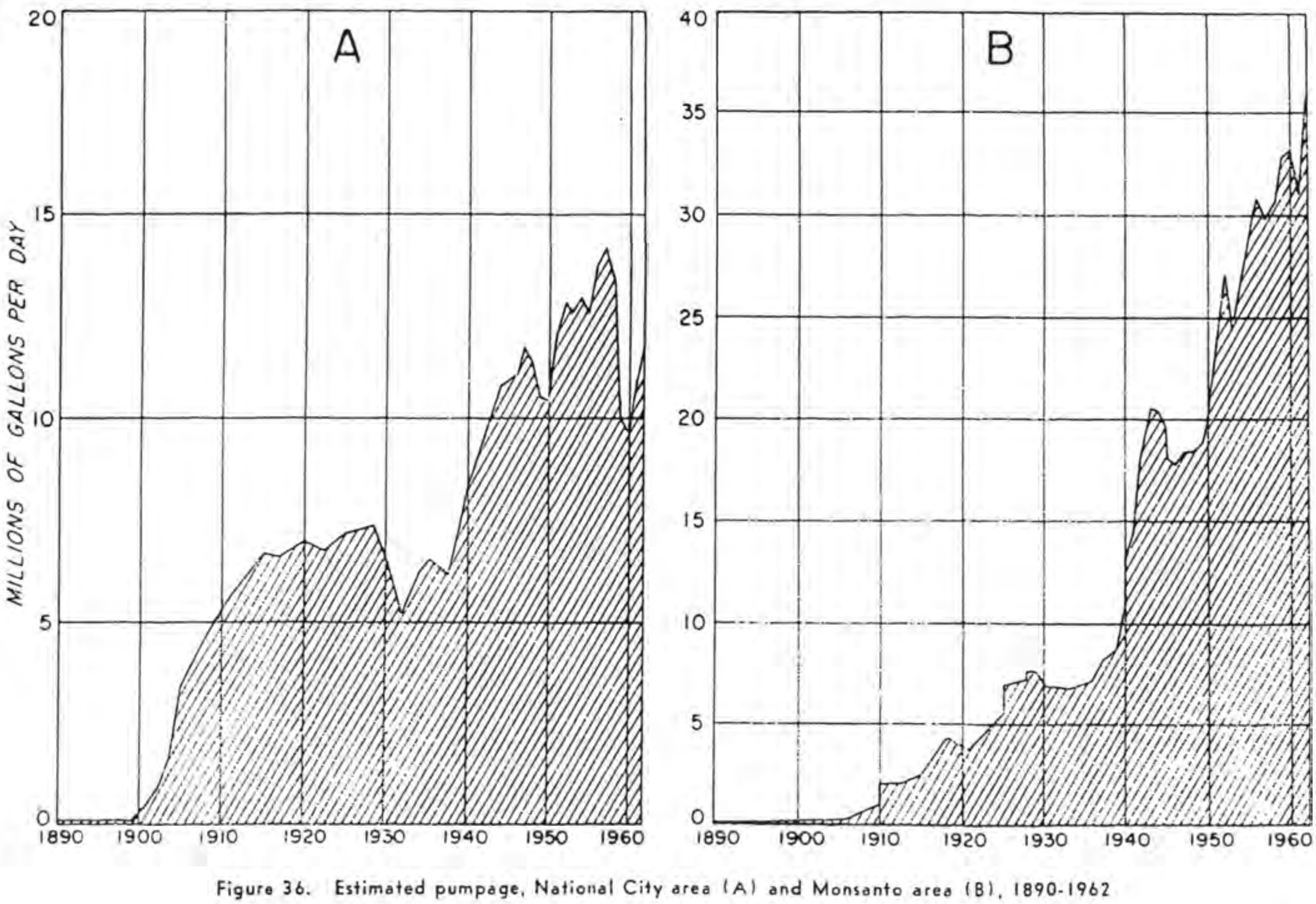


Figure 36. Estimated pumpage, National City area (A) and Monsanto area (B), 1890-1962

fining plants, meat packing plants, and steel plants. Data on industrial pumpage were obtained from 82 plants. Industrial pumpage was 83.5 mgd in 1960, 87.8 mgd in 1961, and 95.7 mgd in 1962. Public supplies include municipal, commercial, and institutional uses. In 1962 there were 10 public water supplies in the East St. Louis area having an estimated total pumpage of 6.7 mgd. Public pumpage was 6.8 mgd in 1960 and 6.6 mgd in 1961. Water pumped by hotels, hospitals, theaters, motels, and restaurants is classified as commercial and institutional pumpage and in 1962 averaged about 400,-000 gpd.

Domestic pumpage, including rural farm nonirrigation and rural nonfarm use, was estimated by considering rural population as reported by the U.S. Bureau of the Census and by using a per capita use of 50 gpd. Domestic pumpage was estimated to be 2.4 mgd in 1960, 1961, and 1962.

Development of ground water for irrigation on a significant scale started in 1954 during the drought extending from 1952 through 1956. In 1962 there were 31 irrigation wells in the East St. Louis area. Estimated irrigation pumpage was 300,000 gpd in 1960, 100,000 gpd in 1961, and 200,000 gpd in 1962.

Prior to 1953 pumpage from wells was largely concentrated in areas at distances of 1 mile or more from the Mississippi River. During and after 1953 pumpage from wells at distances within a few hundred feet from the river increased greatly in the Alton, Wood River, and Monsanto areas. Distribution of pumpage from wells near the river during 1956, 1960, 1961, and 1962 is given in table 22. The distribution of pumpage from wells near the river in 1962 is shown in figure 38. During 1962 total pumpage from Alton, Wood River, and Monsanto area pumping centers was 74.8 mgd of which 31.2 mgd or 41.7 percent was withdrawn from wells near the Mississippi River.

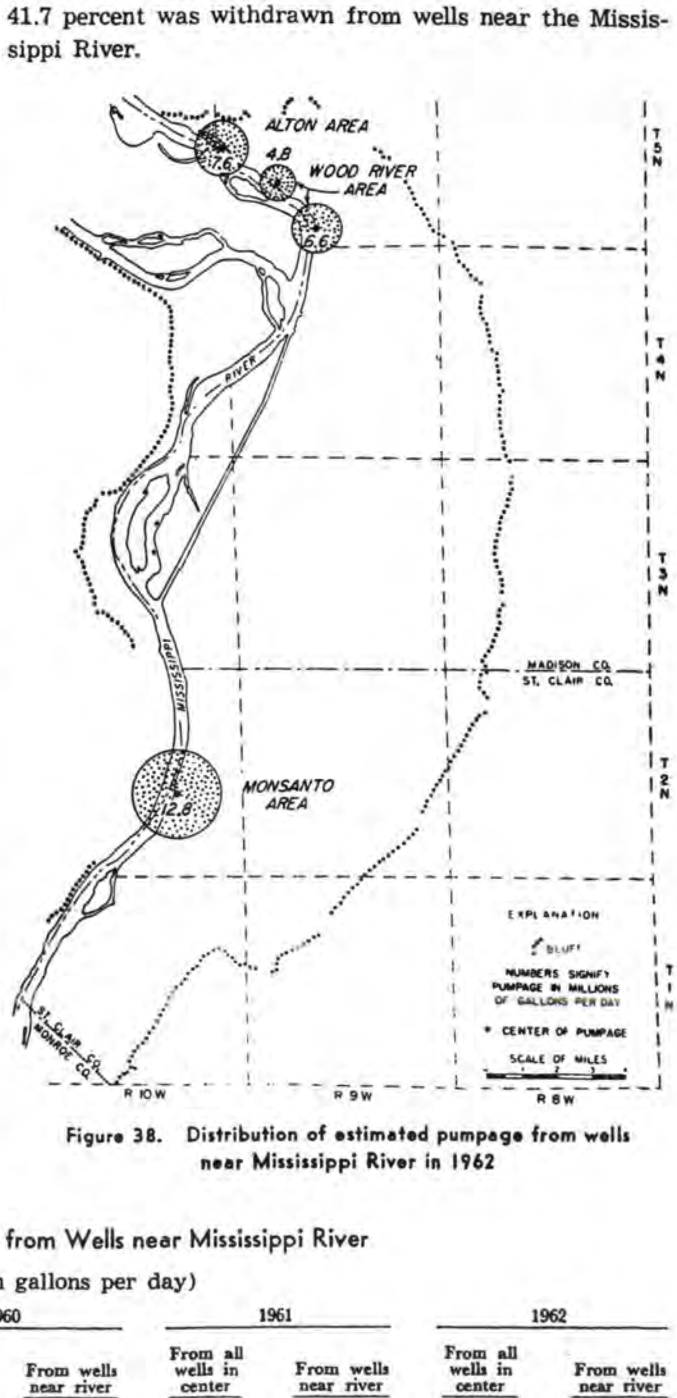


Figure 38. Distribution of estimated pumpage from wells near Mississippi River in 1962

Table 22. Distribution of Pumpage from Wells near Mississippi River

(Pumpage in million gallons per day)

Pumping center	1956 From all wells in center	1956 From wells near river	1960 From all wells in center	1960 From wells near river	1961 From all wells in center	1961 From wells near river	1962 From all wells in center	1962 From wells near river
Alton area	9.8	0	13.6	6.3	12.3	7.2	13.9	7.6
Wood River area	21.1	7.3	20.9	6.8	24.3	10.8	25.5	10.8
Monsanto area	30.1	10.8	33.2	10.5	31.9	11.4	35.4	12.8
Total	61.0	18.1	67.7	23.6	68.5	29.4	74.8	31.2

WATER-LEVEL FLUCTUATIONS

Prior to the settlement of the East St. Louis area, the water table was very near the surface and shallow lakes, ponds, swamps, and poorly drained areas were widespread. Development of the East St. Louis area led to the construction of levees and drainage ditches and subsequent changes in ground-water levels. Bruin and Smith (1953) estimated that these developments caused lowering of ground-water levels by 2 to 12 feet. In addition, industrial and urban expansion and the subsequent use of large quantities of ground water has lowered water levels appreciably in the Alton, Wood River, Granite City, National City, East St. Louis, and Monsanto areas. Lowering of water levels caused by large withdrawals of ground water has also been experienced in the Poag, Caseyville, Glen Carbon, Troy, and Fairmont City areas.

Figure 39 shows the change in water levels in the East St. Louis area during 61 years. The map is based on piezometric surface maps for 1900 and 1961. The greatest declines occurred in the five major pumping centers; 50 feet in the Monsanto area, 40 feet in the Wood River area, 20 feet in the Alton area, 15 feet in the National City area, and 10 feet in the Granite City area. Water levels rose more than 5 feet along Chain of Rocks Canal behind the locks of the canal where the stage of surface water in 1961 was above the estimated piezometric surface in 1900. In areas remote from major pumping centers and the Mississippi River, water levels declined an average of about 5 feet. Water levels

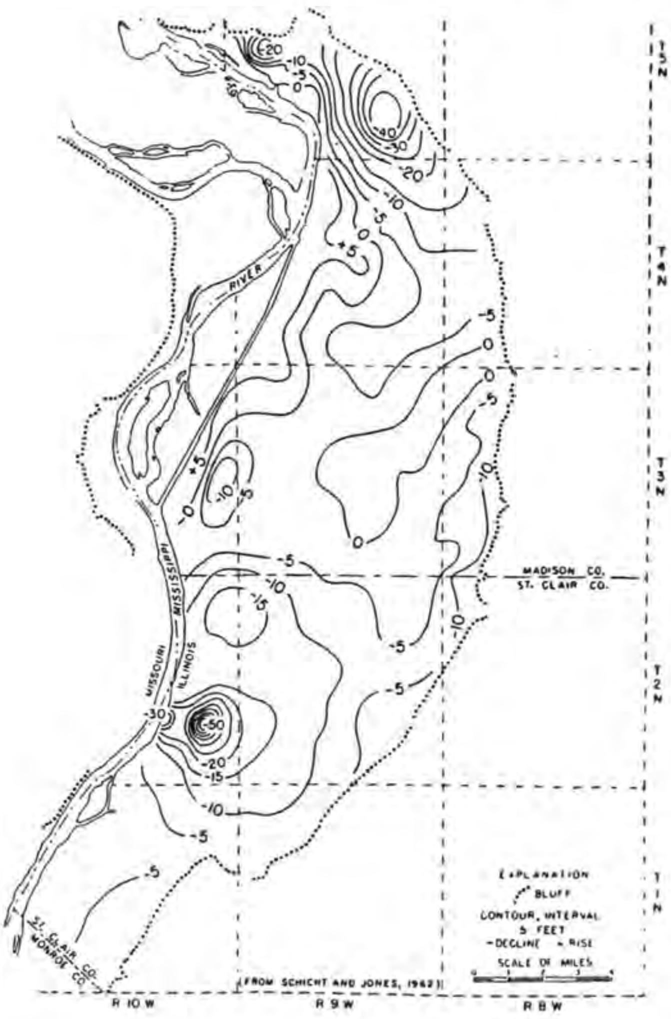


Figure 39. Estimated change in water levels, 1900 to November 1961

have not changed appreciably in the Horseshoe Lake area.

The piezometric surface map for December 1956 was compared with the piezometric surface map for November 1961, and figure 40 shows the change in water levels in the East St. Louis area during this time. The greatest rises in water levels, exceeding 50 feet, were recorded in the Granite City area and are due largely to a reduction in pumpage in the area from 31.6 mgd in 1956 to about 8.0 mgd in 1961. Water levels declined slightly in the center of the Monsanto cone of depression because of an increase in pumpage of about 3 mgd from 1956 to 1961. Water levels rose more than 5 feet in other places in the Monsanto area and more than 10 feet in the Alton area. Water levels in the Wood River area declined less than 1 foot near the center of pumping and rose more than 10 feet in other places. Along the Mississippi River west of Wood River water levels rose more than 20 feet; along the Mississippi River west of Monsanto water levels declined slightly in an area affected by an increase in pumpage from wells near the river. In areas remote from major pumping centers and the Mississippi River, water levels rose on the average about 5 feet.

Changes in water levels from June to November 1961 were computed (Schicht and Jones, 1962) and were used to prepare figure 41. The stage of the Mississippi River was higher during November than in June, and as a result ground-water levels rose appreciably along the river especially in areas where induced infiltration occurs. Water levels declined more than a foot at many places in the Granite City and National City areas and along the bluffs north of Prairie Du Pont Creek. Water-level declines averaged about 3 feet south of Prairie Du Pont Creek. Water-level rises exceeded 5 feet in the Alton area and exceeded 7 feet along the Mississippi River west of Wood River. Water levels rose in excess of 4 feet in the Monsanto area. A tongue of water-level rise extended eastward through Monsanto and to a point about 5 miles northeast of Monsanto.

Changes in water levels from June 1961 to June 1962 are shown in figure 42. The stage of the Mississippi River was higher during June 1962 than in June 1961, and as a result ground-water levels rose appreciably in most places along the Mississippi River and Chain of Rocks Canal. Water levels declined more than a foot near Monsanto along the Mississippi River as a result of heavy pumping. Water levels declined less than a foot in the Horseshoe Lake area and in places along the bluffs; water levels also declined in a strip west of Dupo. Water levels rose in excess of 5 feet along the Mississippi River in the Alton and Wood River areas and along the northern reach of Chain of Rocks Canal. Immediately east of Dupo water levels rose in excess of 4 feet.

Changes in water levels from November 1961 to June 1962 are shown in figure 43. Ground-water levels rose appreciably in most places because Mississippi

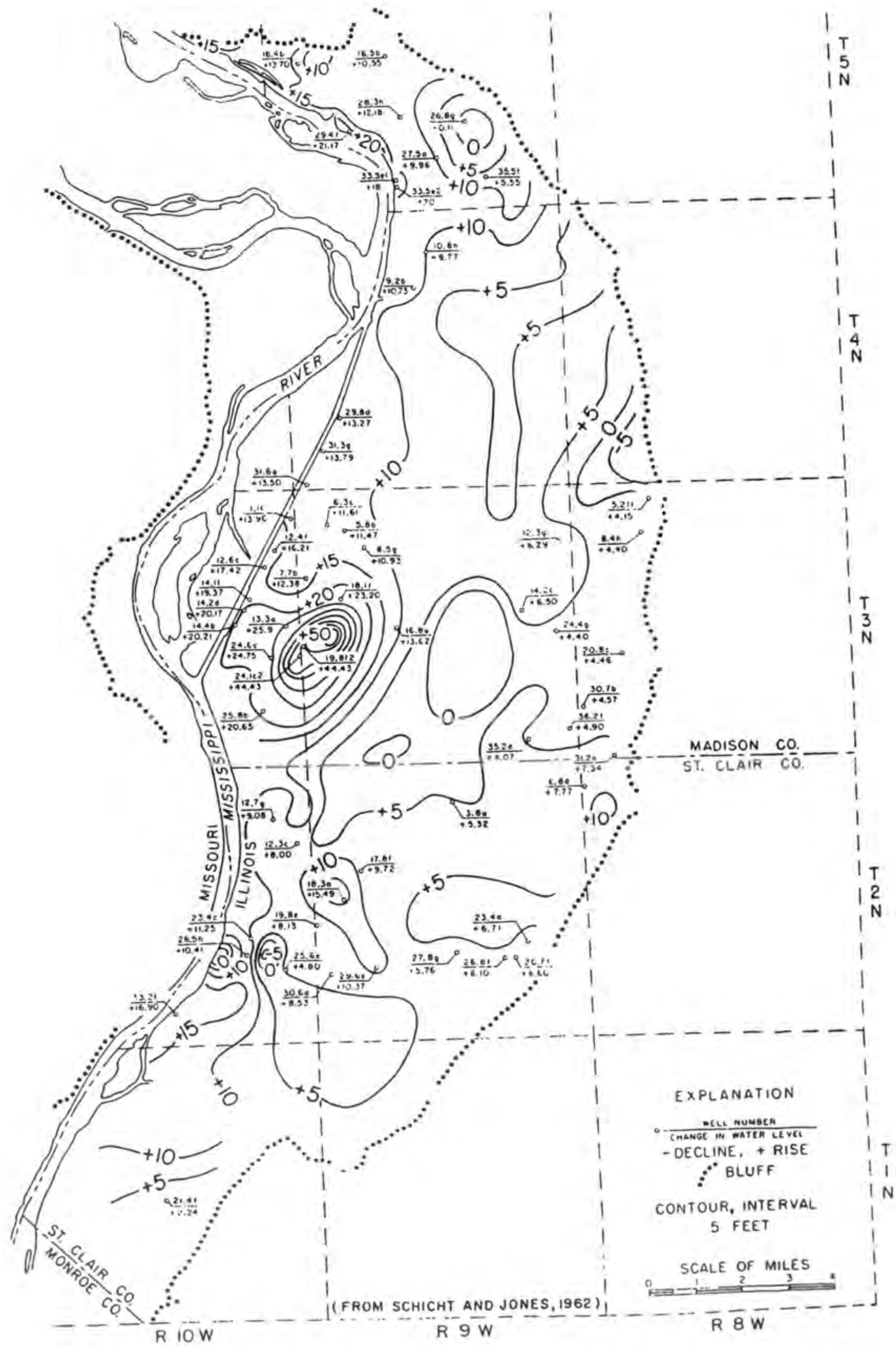


Figure 40. Estimated change in water levels, December 1956 to November 1961

River stages were higher in June 1962 than in November 1961. During the winter and early spring months, conditions were favorable for the infiltration of rainfall to the water table. Ground-water levels rose appreciably along the bluffs, the rise exceeding 7 feet in places. Ground-water level rises along the Mississippi River exceeded 5 feet east of Wood River and east of National City; ground-water level rises exceeded 5 feet at the northern end of Long Lake and near Dupo. Water levels declined less than 1 foot around Horseshoe Lake and between 1 and 2 feet in a small area near Monsanto.

Examples of fluctuations in water levels in the East St. Louis area are shown in figures 44-49. The locations of observation wells for which hydrographs are available are given in figure 50. As illustrated by the hydrographs for wells remote from major pumping centers in figure 44, water levels generally recede in the late spring, summer, and early fall when discharge from the ground-water reservoir by evapotranspiration, by ground-water runoff to streams, and by pumping from wells is greater than recharge from precipitation and induced infiltration of surface water from the Mississippi River and other

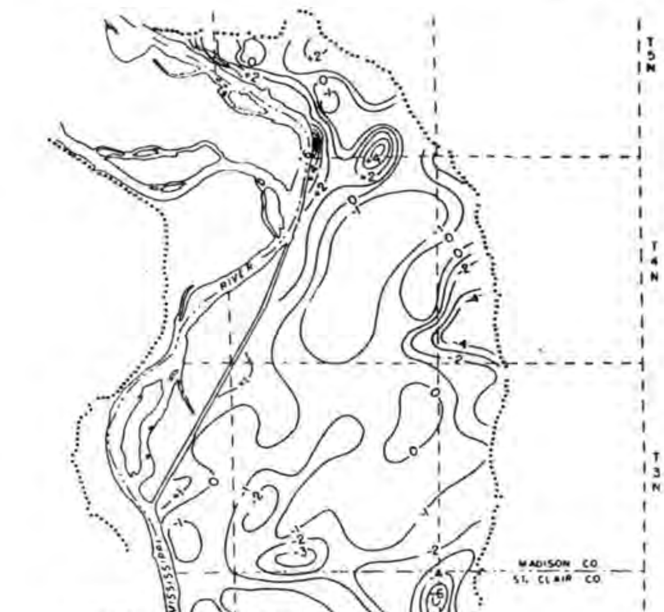


Figure 41. Estimated change in water levels, June to November 1961

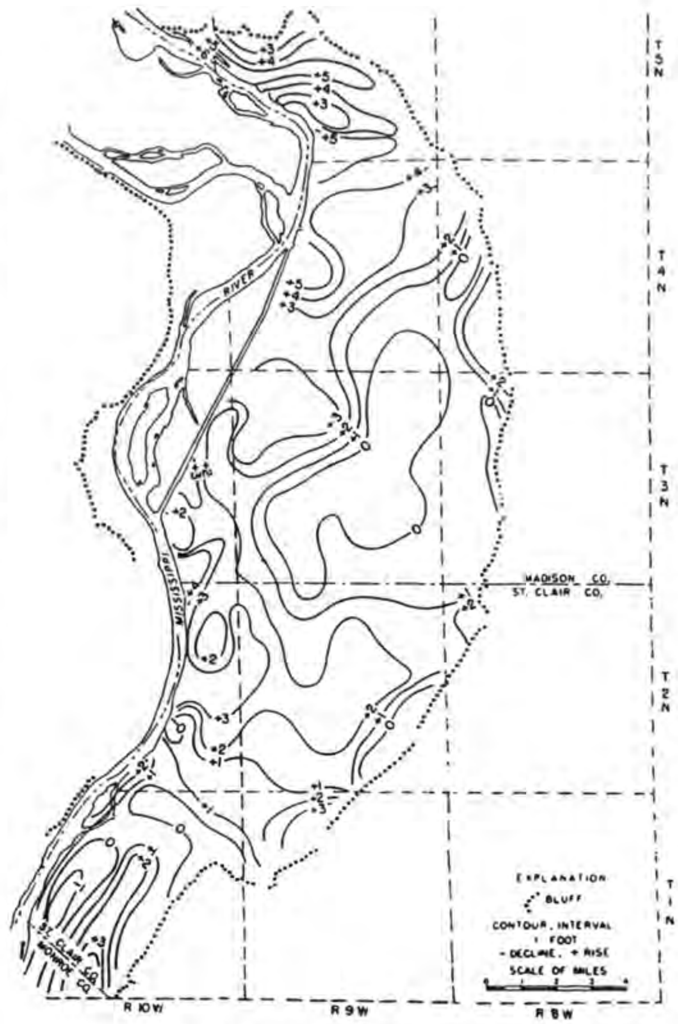


Figure 42. Estimated change in water levels, June 1961 to June 1962

streams. Water levels generally begin to recover in the early winter when conditions are favorable for the infiltration of rainfall to the water table. The recovery of water levels is especially pronounced during the spring months when the ground-water reservoir receives most of its annual recharge. Water levels are frequently highest in May and lowest in December, depending primarily upon climatic conditions, pumping rates, and the stage of the Mississippi River. Water levels in wells remote from major pumping centers have a seasonal fluctuation ranging from 1 to 13 feet and averaging about 4 feet.

Water levels in the East St. Louis area declined appreciably during the drought, 1952-1956. The records of the U.S. Weather Bureau at Edwardsville indicate that rainfall averaged about 34.3 inches per year from 1952 through 1956, or about 6.5 inches per year below normal. The hydrograph of water levels in well MAD 3N8W-31.2a and the graph of annual precipitation at Edwardsville for 1941 to 1962 in figure 45 illustrate the pronounced effect of the prolonged drought on water levels.

Examples of hydrographs of water in wells within major pumping centers are shown in figures 46-49. Comparisons of pumpage and water-level graphs indicate that in general water levels within pumpage centers fluctuate in response to changes in precipitation, river stage, and pumpage. The effects of the drought during 1952-1956 are apparent; the effects of changes in river stage are masked almost completely by the effects of the drought and pumpage changes. However, careful study of river stages and water-level data indicate that water levels in major pumpage centers do fluctuate several feet in response to large changes in river stage. If the effects

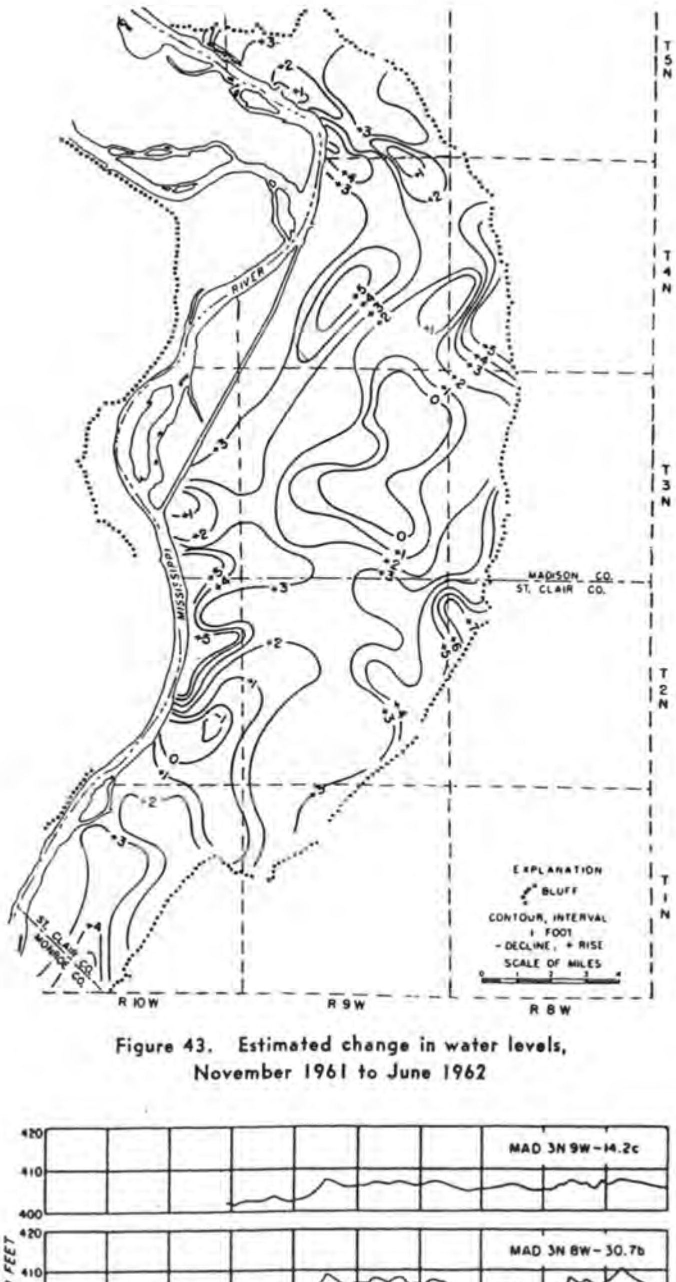


Figure 43. Estimated change in water levels, November 1961 to June 1962

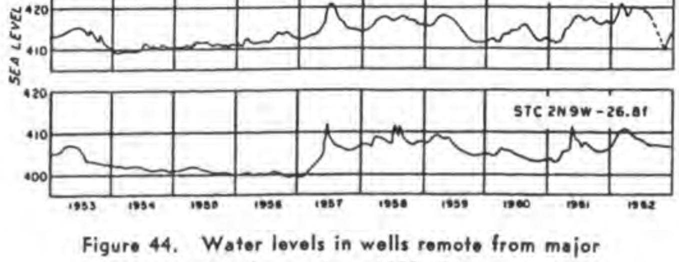


Figure 44. Water levels in wells remote from major pumping centers, 1953-1962

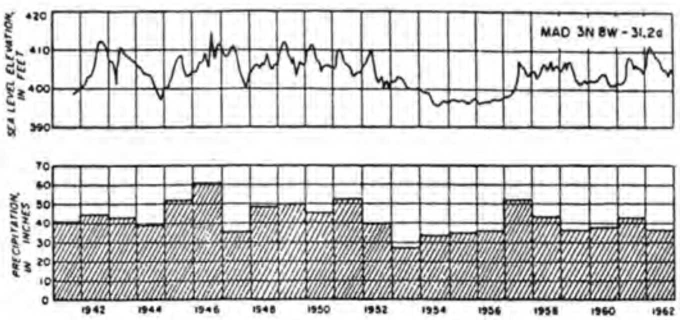


Figure 45. Water levels in well MAD 3N8W-31.2a and annual precipitation at Edwardsville, 1941-1962

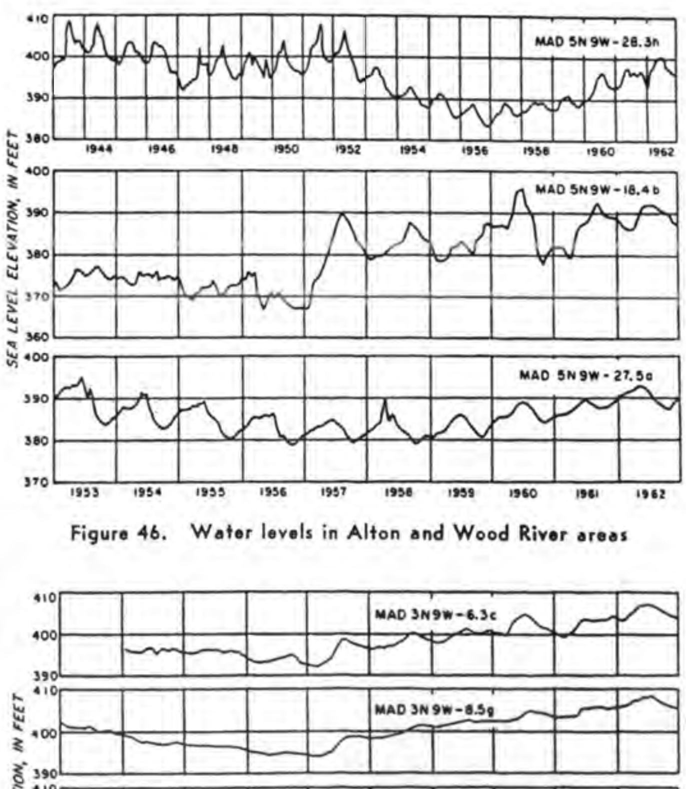


Figure 46. Water levels in Alton and Wood River areas

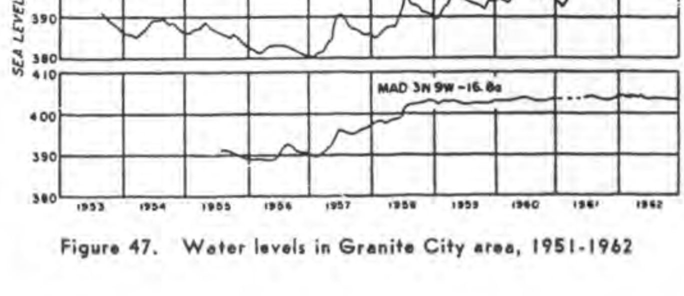


Figure 47. Water levels in Granite City area, 1951-1962

of the drought and changes in river stage are taken into consideration, water-level declines are directly proportional to pumping rates. The water levels vary from place to place within pumpage centers and from time to time mostly because of the shifting of pumpage from well to well, shifting of pumpage from pumpage centers 1 mile or more from the Mississippi River to pumpage centers near the river, and variations in total well field pumpage. At no location is there any apparent continuous decline that cannot be explained by pumpage increases. Thus, within a relatively short time after each increase in pumpage, recharge directly from precipitation and by induced infiltration of water in streams increased in proportion to pumpage as hydraulic gradients became greater and areas of diversion expanded.

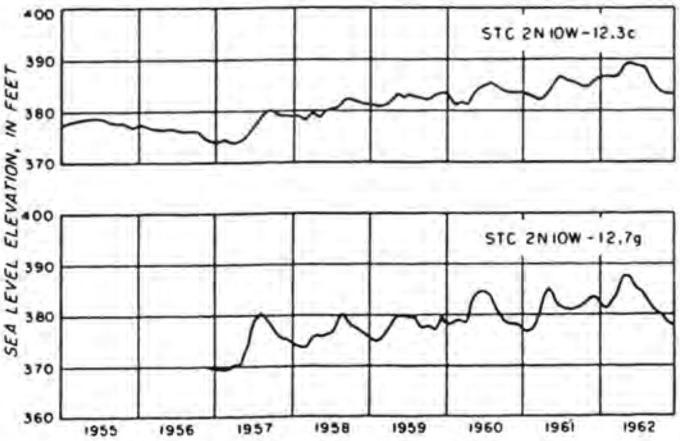


Figure 48. Water levels in wells in National City area, 1955-1962

Annual fluctuations of water levels in wells within major pumping centers are generally less than 15 feet. The average rate of decline during 1952-1956 was about 2 feet per year. The average rate of rise in the Granite City area during the period 1957-1962 was about 2 feet per year. The average rate of decline in the Monsanto area during 1930-1962 was about 1.3 feet per year.

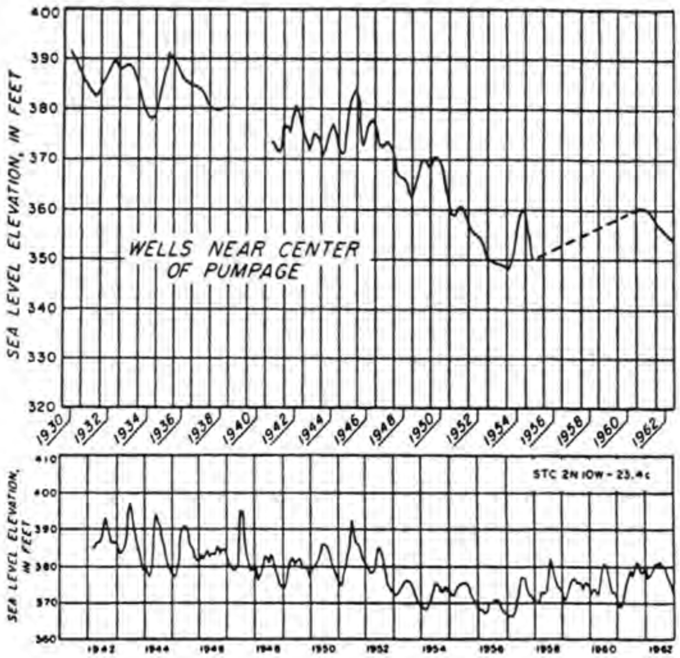


Figure 49. Water levels in wells in Monsanto area

PIEZOMETRIC SURFACE

In order to delineate areas of diversion and to determine directions of ground-water movement in the East St. Louis area, piezometric surface maps were made.

Figure 51 depicts the surface drainage system in 1900 and the estimated piezometric surface prior to heavy industrial development. The piezometric surface sloped from an estimated elevation of about 420 feet near the bluffs to about 400 feet near the Mississippi River. The average slope of the piezometric surface was about 3 feet per mile; however, the slope ranged from 6 feet per mile in the Alton area to 1 foot per mile in the Dupo area. The slope of the piezometric surface was greatest near the bluffs. The general direction of ground-water movement was west and south toward the Mississippi River and other streams and lakes. The establishment of industrial centers and the subsequent use of large quantities of ground water by industries and municipalities has lowered water levels appreciably in the areas of heavy pumping.

From 1952 through 1956 water levels declined appreciably in the East St. Louis area as the result of drought conditions, low Mississippi River stages, and record high ground-water withdrawals. Figure 52 shows the piezometric surface in December 1956, when water levels were at record low stages at many places.

The illustration shows clearly the cones of depression in the piezometric surface which have developed as the result of heavy pumping. It will be noted that a considerable lowering has taken place in the piezometric surface since 1900. In 1956 the deepest cone of depression was in the Granite City area. Other pronounced cones were centered in major pumping centers.

Figure 53 shows the piezometric surface in June 1961 after pumpage was reduced in the Granite City area. The piezometric surface map for December 1956 is similar in many respects to the piezometric surface map for June 1961. Significant differences are that the cone of depression in the Granite City area was much deeper

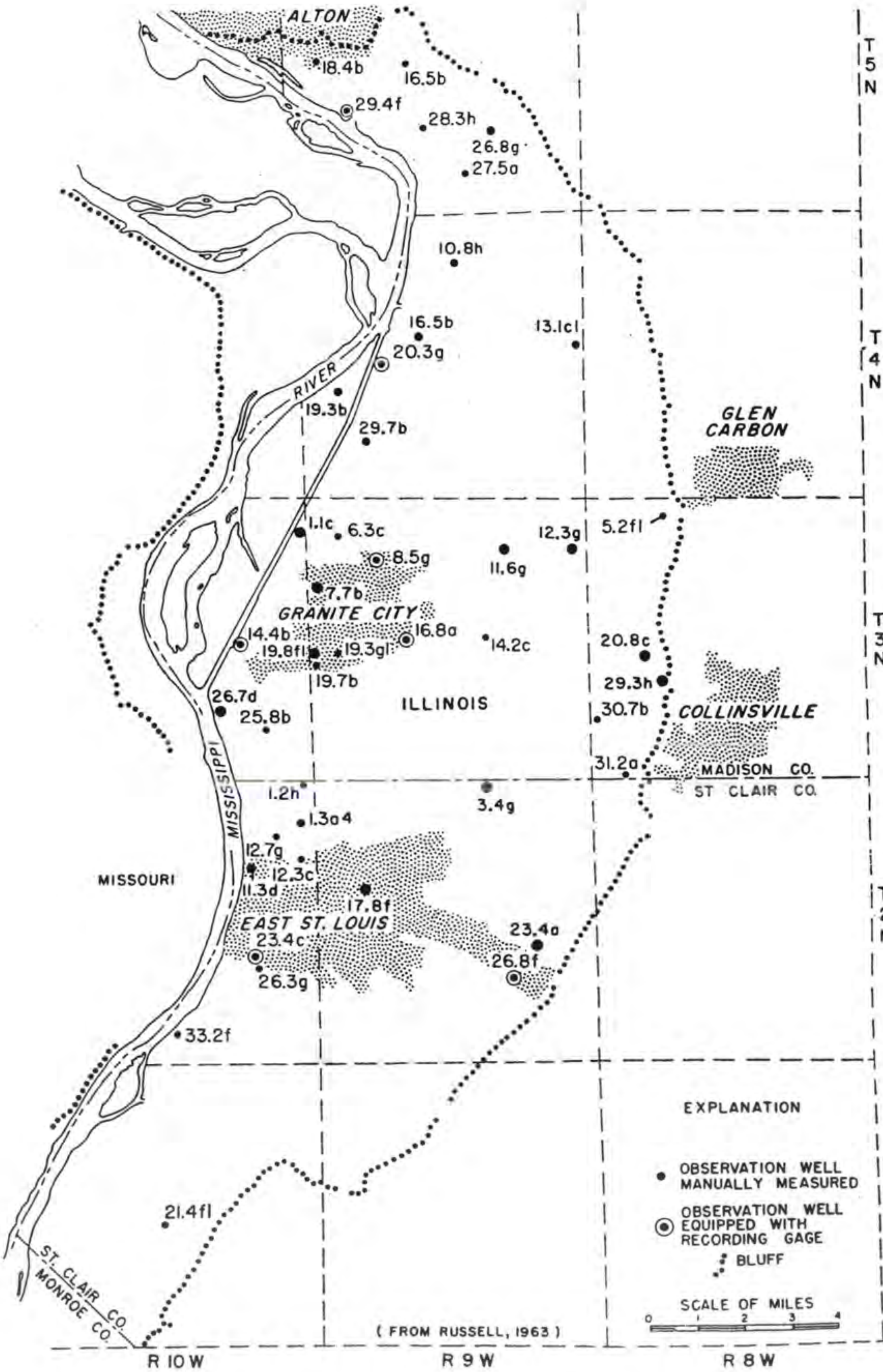


Figure 50. Locations of observation wells

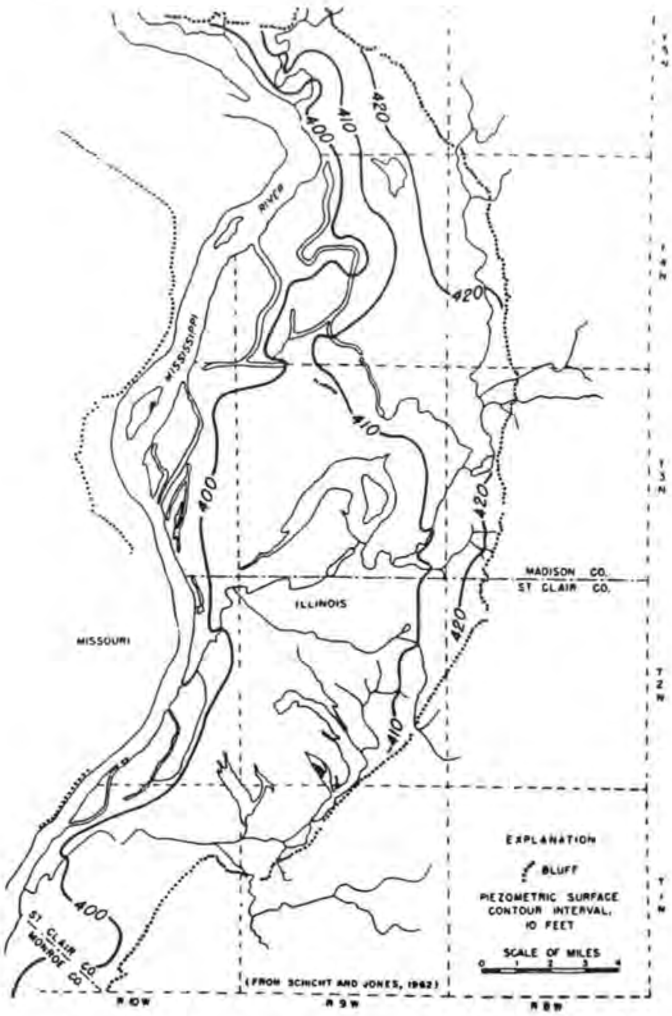


Figure 51. Drainage system and estimated elevation of piezometric surface about 1900

in 1956 than in 1961, and ground-water levels were lower in the vicinity of streams and lakes in 1956 than they were in 1961.

During June 1962, when water levels were near peak stages, a mass measurement of ground-water levels was made, and data collected are given in tables 23, 24, and 25. The piezometric surface map for June 1962 is shown in figure 54. Features of the piezometric surface maps for June 1961 and June 1962 are generally the same. The deepest cone of depression in June 1962 was centered in the Monsanto area where the lowest water levels were at an elevation of about 350 feet. A smaller cone of depression occurred near the Mississippi River about 1.5 miles west of the large Monsanto cone of depression in the vicinity of a small pumping center. The water levels in the center of this cone of depression were at an elevation of about 355 feet. The elevations of the lowest water levels in other important cones of depression were: 385 feet in the Wood River area, 390 feet in the Alton area, 395 feet in the Granite City area, and 390 feet in the National City area.

The general pattern of flow of water in 1962 was slow movement from all directions toward the cones of depressions or the Mississippi River and other streams. The lowering of water levels in the Alton, Wood River, National City, and Monsanto areas that has accompanied withdrawals of ground water in these areas has established hydraulic gradients from the Mississippi River towards pumping centers. Ground-water levels were below the surface of the river at places and appreciable quantities of water were diverted from the river into the aquifer by the process of induced infiltration. The piezometric surface was above the river at many places. For example, southwest of the Granite City cone of depression water levels adjacent to the river were higher than the normal river stage and there was discharge of ground water into the river.

The average slope of the piezometric surface in areas remote from pumping centers was 5 feet per mile. Gradients were steeper in the immediate vicinity of major

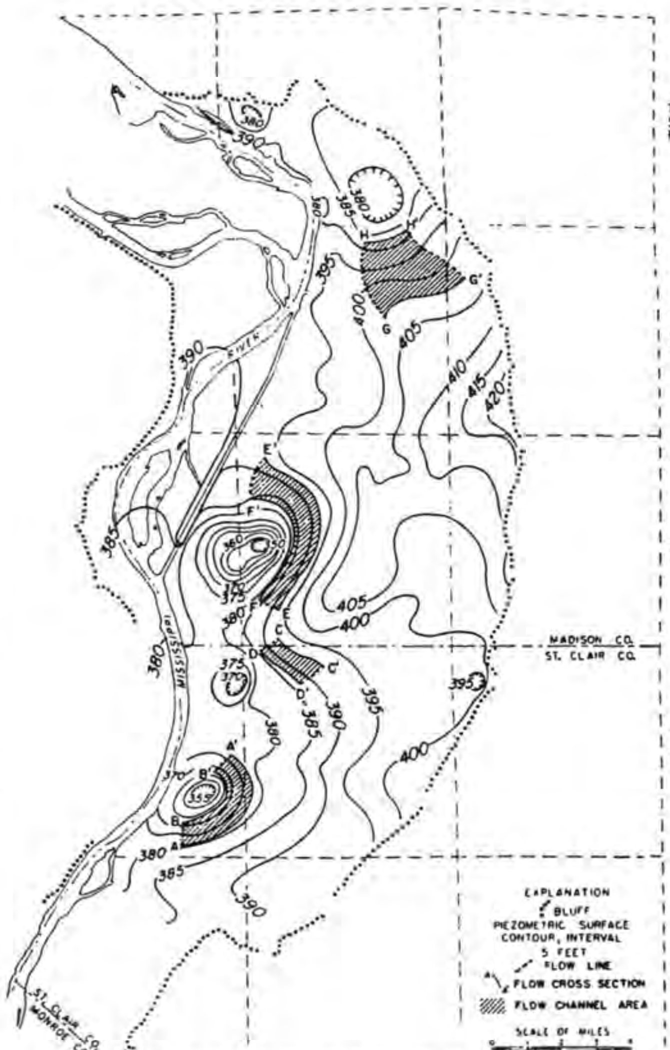


Figure 52. Approximate elevation of piezometric surface, December 1956

Table 23. Water-Level Data for Wells

Well number	Elevation of measuring point (ft)	Water levels, June 1962: Depth to water (ft)	Water levels, June 1962: Mean sea level elevation (ft)	Water level changes (ft): From June 1961 to June 1962	Water level changes (ft): From November 1961 to June 1962
MAD—					
5N10W-					
13.2a	413.4	7.52	405.88	+5.28	+3.11
13.5c	412.8	6.44	406.36	+5.88	+1.56
13.6cl	416.1	9.42	406.68	+5.40	+1.25
13.7e	415.2	6.32	408.88	+5.60	+0.14
14.1e	411.9	1.55	410.35	+6.37	+0.54
24.1h	414.7	9.31	405.39	+5.19	+2.96
5N9W-					
16.5b	443.03	44.25	398.78	+4.42	+2.33
18.3c	436.7	49.11	387.59	+2.91	+3.19
18.4b	438.1	47.10	391	+3.00	+2.00
19.3c	415.7	7.56	408.14	+5.20	+2.54
19.4h	430	35.45	394.55	+3.20	+2.09
19.6e	415.8	7.97	407.83	+5.79	+2.67
19.7f	413.3	6.44	406.86	+5.77	+3.03
19.8g	414.7	8.02	406.68	+5.80	+3.20
20.5a	413.4	5.68	407.72	+3.18	+1.05
22.2c	440.71	47.57	393.14	+5.53	+3.99
26.8g	441.42	57.17	384.25	+4.53	+2.86
27.5a	428.52	36.25	392.27	+2.96	+3.61
28.3h	432.60	32.25	400.35	+3.20	+3.77
28.4c	413.30	11.22	402.08	+2.85	+1.09
28.8e	418.2	10.29	407.91	+4.75	+1.29
29.1e	413.4	11.15	402.25	+4.40	+1.07
29.4f	416.07	8.67	407.40	+2.68	+0.33
29.4g	414.4	7.30	407.10	+3.61	+0.83
29.5g	415.4	8.26	407.14	+3.41	+0.54
33.5el	418.44	16.00	402.44	+11.00	+5.00
33.5e2	417.89	14.00	403.89	+10.00	+3.00
34.7c	431	37.20	393.80		+3.00
35.5f	445.55	60.55	385	+5.80	+3.45
35.5h	446.53	64.53	382	+3.50	+3.47
35.6b	445.69	50.69	395	+4.81	+0.52
4N8W-					
6.8h	441.18	31.45	409.76	+3.42	+2.28
7.4b	428	18.06	409.94	+2.03	+3.02
19.4e	429	16.86	412.14	−0.42	+0.91
20.3c	452.5	30.32	422.18	+9.12	+11.48
30.1f	425	8.07	416.93	+2.38	+6.88
4N9W-					
9.2b	434.61	25.05	409.56	+3.13	+2.33
10.8h	432.57	24.8	407.77	+4.00	+3.00
12.4h	427	19.65	407.35	+2.79	
13.1c1	439.15	26.00	413.15	+2.00	
14.8h	422.89	16.50	406.39		+3.34
16.5b	417.78	7.45	410.33		
19.3b	422	18.49	403.51	+2.30	+0.46
20.3g	414.39	5.40	408.99	+5.18	+1.25
25.2d	421	8.73	412.27	+0.60	+0.32
27.8h	409	1.00	408.00	+3.66	+5.46
29.7b	421.06	13.17	407.89		+1.83
29.8d	413.42	5.58	407.84	+2.27	+2.05
30.1b	416.70	8.88	407.82	+2.53	+1.90
31.2h	416.95	9.28	407.67	+2.72	+1.86
31.3g	415.57	7.98	407.59	+2.77	+1.69
31.6a	408.02	1.09	406.93	+2.76	+1.61
34.1h	423	14.84	408.16	+0.76	+1.87
34.5a	421	10.36	410.64		+1.87

Well number	Elevation of measuring point (ft)	Water levels, June 1962: Depth to water (ft)	Water levels, June 1962: Mean sea level elevation (ft)	Water level changes (ft): From June 1961 to June 1962	Water level changes (ft): From November 1961 to June 1962
4N10W-					
36.5f	415	2.75	412.25	−0.77	+1.89
3N8W-					
5.2f1	439.65	20.00	419.65	+2.00	+3.00
6.1e	425	6.75	418.25	+0.43	+2.01
8.4h	430	10.78	419.22	−0.24	+1.82
8.8a	422	9.35	412.65	+0.68	+1.97
17.4d	416.06	4.02	412.04	−0.02	+0.76
20.5c	430	19.46	411.54		+4.10
20.8c	422	10.01	411.99	+0.69	+2.28
30.7b	421.28	12.55	408.73	+0.52	+2.16
31.2a	428.22	20.04	408.18	+0.11	+3.44
3N9W-					
3.1a	415	7.05	407.95	−0.42	−0.19
5.8b	424.45	16.78	407.67	+3.91	+3.20
6.3c	426.66	19.39	407.27	+3.54	+2.96
7.7b	425.08	19.15	405.93	+3.57	+7.95
8.5g	420.84	12.40	408.44	+3.27	+2.91
9.4e	421	12.80	408.20	+3.67	+4.34
10.4b	415	5.35	409.65	+0.26	+1.31
12.3g	420.5	4.76	415.74	−0.18	−0.55
14.2c	425.50	17.36	408.14	−0.09	−0.36
16.1d	422	14.82	407.18	−0.63	+1.23
16.8a	415.88	10.87	405.01	−0.11	+0.89
18.1f	412.90	8.88	404.02	+2.42	+2.42
19.3g1	417.74	15.78	401.96		
19.8f1	424.14	26.68	397.46		+6.74
20.2h	414.67	9.54	405.13	+0.18	+0.82
20.7e	418.73	14.87	403.86	−0.10	+1.57
20.8d1	416.68	13.57	403.11	−0.13	+2.10
20.8d2	414.71	11.89	402.82	−0.16	+1.86
20.8e1	416.33	13.03	403.30	−0.12	+1.82
21.2d	408	4.85	403.15	−0.91	−0.69
23.5g	419	16.65	402.35	−0.45	+0.05
24.4g	425.90	16.55	409.35	+0.38	+1.15
28.4e	417.5	6.81	410.69	+0.27	+1.34
32.3b	410	12.15	397.85	−0.21	+3.21
32.6g	418	17.58	400.42		
35.2d	411.21	6.34	404.87	+0.31	+1.59
35.5g	415.5	8.65	406.85	+0.33	+0.95
36.2f	421.12	13.07	408.05	+0.59	+2.15
3N10W-					
1.1c	407.11	0.00	407.11	+3.22	+2.05
12.4f	406.98	0.73	406.25	+1.86	+1.25
12.6c	407.51	1.34	406.17	+1.80	+1.34
13.8g	409.43	5.58	403.85	+2.64	+2.72
14.1f	406.78	4.11	402.67	+1.99	+1.84
14.2d	411.36	7.63	403.73	+3.05	+2.56
14.3c	413.53	9.92	403.61	+2.93	+2.44
14.4b	413.69	7.59	406.10	+5.40	+5.09
22.1a	412.2	10.82	401.38	+2.13	+0.94
22.1c	412.9	12.39	400.51	+2.15	+1.45
23.6c	413.5	13.40	400.10	+1.75	+0.68
23.7c	412.4	12.35	400.05	+1.02	+1.35
24.1c1	422.34	24.47	397.87	+1.63	+2.23
24.1c2	418.59	20.97	397.62	+1.58	+2.19
24.6c	420	29.07	390.93	+2.08	+2.18
25.8b	414.96	14.94	400.02	+1.84	+2.57
26.6b	411.3	10.73	400.57	+0.45	+1.80

Table 23 (Continued)

Well number	Elevation of measuring point (ft)	Water levels, June 1962: Depth to water (ft)	Water levels, June 1962: Mean sea level elevation (ft)	Water level changes (ft): From June 1961 to June 1962	Water level changes (ft): From November 1961 to June 1962
3N10W-(Continued)					
26.7d	411.2	10.21	400.98	+1.24	+1.60
26.8e	411.1	10.16	400.94	+1.53	+1.81
26.8h	411.8	10.80	401.00	+1.57	+1.76
35.6f	401.8	0.97	400.80	+4.21	+5.22
35.6h	404.6	4.19	399.60	+1.02	
36.5h	414.25	13.29	400.96	+3.16	+3.81
STC—					
2N8W-					
6.1d	425	17.20	407.80	+2.02	+4.28
6.8d	429.27	15.00	414.27	+1.00	+7.00
7.2h2	430	22.63	407.37		+5.72
2N9W-					
2.4e	418.5	6.85	411.65	+0.95	+3.05
3.4g	422	15.44	406.56	+0.56	+3.25
3.8a	424	23.01	400.99	+1.98	+2.67
7.5e	420	33.60	386.40	+2.98	+2.56
7.6c	420	34.03	385.97	+5.37	
11.7h	419	11.85	407.15	+1.71	+3.53
12.5d	420	7.88	412.12	+0.76	+4.66
13.6c	421.70	12.00	409.70	+1.00	+4.33
14.5c	425	18.38	406.62	+3.17	+3.39
15.3b	413	8.16	404.84	+3.21	+4.74
15.7a	420	18.32	401.68	+2.13	+2.97
17.2d	415	17.60	397.40	+1.76	+0.88
17.8f	417.21	24.12	393.09	+2.43	+1.37
18.3a	416.5	22.60	393.90	+2.89	+1.41
19.8e	418.78	31.24	387.54		+0.81
21.7h	410	15.18	394.82	+2.38	+2.14
23.4a	423.86	10.98	412.88	−1.98	+4.87
24.6e	428	16.42	411.58	−0.89	+4.14
26.7f	424.18	15.24	408.94	−0.07	+2.86
26.8f	421.39	12.78	408.61	−0.01	+2.84
27.8g	415	9.44	405.56		+2.36
28.4g	409	1.55	407.45	+2.15	
30.6d	415	25.53	389.47	+0.88	+1.94
32.2c	408	12.28	395.72	+0.73	+2.58
34.4h	417	12.06	404.94	−0.02	+2.18
1.2h	412	20.15	391.85	+2.42	+3.38
1.3a1	418.4	31.0	387.40		
12.3c	418.54	29.62	388.92	+2.53	+6.92
12.7g	410	23.91	386.09	+1.06	+2.41
23.4c	399.72	19.94	379.78	+1.25	+1.43
23.6f	415.7	23.49	392.21	+3.30	+5.37
2N9W-(Continued)					
23.6g	397.5	4.04	393.46	+3.63	+1.37
23.7a	406.5	24.59	381.91	+0.89	+0.72
23.7b	408.2	20.63	387.57	+0.84	+1.97
26.1g1	411.37	72.50	338.87		
26.1g2	411.24	65.50	345.74	+2.50	−1.90
26.2e	413.70	61.67	352.03		−0.83
26.3g	411.80	55.33	356.47		
26.5h	408.76	34.30	374.46	+1.22	+0.15
27.2h1	415.65	62.25	353.40	−9.85	
33.2f	409.35	13.11	396.24	+2.94	+2.34
34.7c	399.1	5.25	393.85	+0.14	+2.52
34.8b	398.0	3.58	394.42	−0.08	+2.60
1N9W-					
4.5e	411	8.49	402.51	+0.84	+3.42
6.2a	416	18.43	397.57	−0.39	+2.20
1N10W-					
4.1g	399.0	3.99	395.01	−0.49	+2.15
4.2e	396.4	1.04	395.36	−0.34	+1.76
4.3b	398.6	2.95	395.65	−0.17	+1.08
4.3c	397.7	2.27	395.43	+0.50	+1.10
4.7b	409.4	12.85	396.55	+0.95	+2.47
8.2h	407.8	11.11	396.69	+0.72	+2.76
8.5c	405.1	8.27	396.83	−0.27	+3.64
8.7a	406.3	9.89	396.41	−0.23	+2.00
9.1f	403.63	5.65	397.98	+0.84	+1.97
9.2h	404.55	6.94	397.61	+0.52	+1.68
9.4h	409.9	13.93	395.97	+0.78	+1.83
10.1c	403.29	5.14	398.15	+0.94	+1.94
10.4c	402.24	4.23	398.01	+0.82	+1.89
12.5b	401.74	3.09	398.65	−0.23	+0.95
13.3h	402.25	3.43	398.82	−0.32	+0.68
16.2g	411.5	10.96	400.54	+2.66	+3.26
17.1e	400	3.75	396.25	−0.71	+3.67
19.6f	406.4	10.21	396.19	−1.03	+2.52
21.1a	410	13.63	396.37	+0.68	−3.38
21.4f	412.01	13.85	398.16		+2.92
28.6a	405	9.54	395.46	+0.38	+2.93
30.6h	405.3	9.30	396.00	−0.99	+2.54
32.3e	414	18.81	395.91	+4.19	+5.10
MON—					
1N10W-					
30.8b	408.1	12.01	396.01	−0.85	+2.34
31.4d	407	10.55	396.45	+1.14	+3.30

pumping centers and exceeded 30 feet per mile within the Monsanto cone of depression. Gradients averaged about 10 feet per mile within the Alton, Granite City, National City, and Wood River cones of depression.

Along Canteen Creek and Cahokia Canal east of Horseshoe Lake, Long Lake, and Grand Marais State Park Lake, the piezometric surface was higher than the surface-water elevation and ground water was discharged into these streams and lakes. Below the confluence of Canteen Creek and Cahokia Canal south of Horseshoe Lake the piezometric surface was lower than surface-water elevations of Cahokia Canal at places where water levels have declined as the result of heavy pumping. Surface water in the Cahokia Diversion Channel south of the Wood River is kept above the piezometric surface at an elevation of 413 feet by a low water dam near the outlet of the channel. Surface-water levels are also controlled in Chain of Rocks Canal by Lock No. 27 near Granite City and were higher than the piezometric surface adjacent to the canal. The piezometric surface in the vicinity of Wood River near Alton and Prairie Du Pont Creek south of Monsanto was slightly higher than the surface-water elevations of the streams. At the lower end of Horseshoe Lake north of National City, ground-water levels were lower than the surface-water elevation of the lake.

South of Prairie Du Pont Creek ground water normally flows toward the Mississippi River. Ground water flows from the vicinity of Long Lake northwest towards the Mississippi River between the northern end of Chain of Rocks Canal and the outlet of the Cahokia Diversion Channel. Ground water flows toward the Mississippi River along the western half of Chouteau Island.

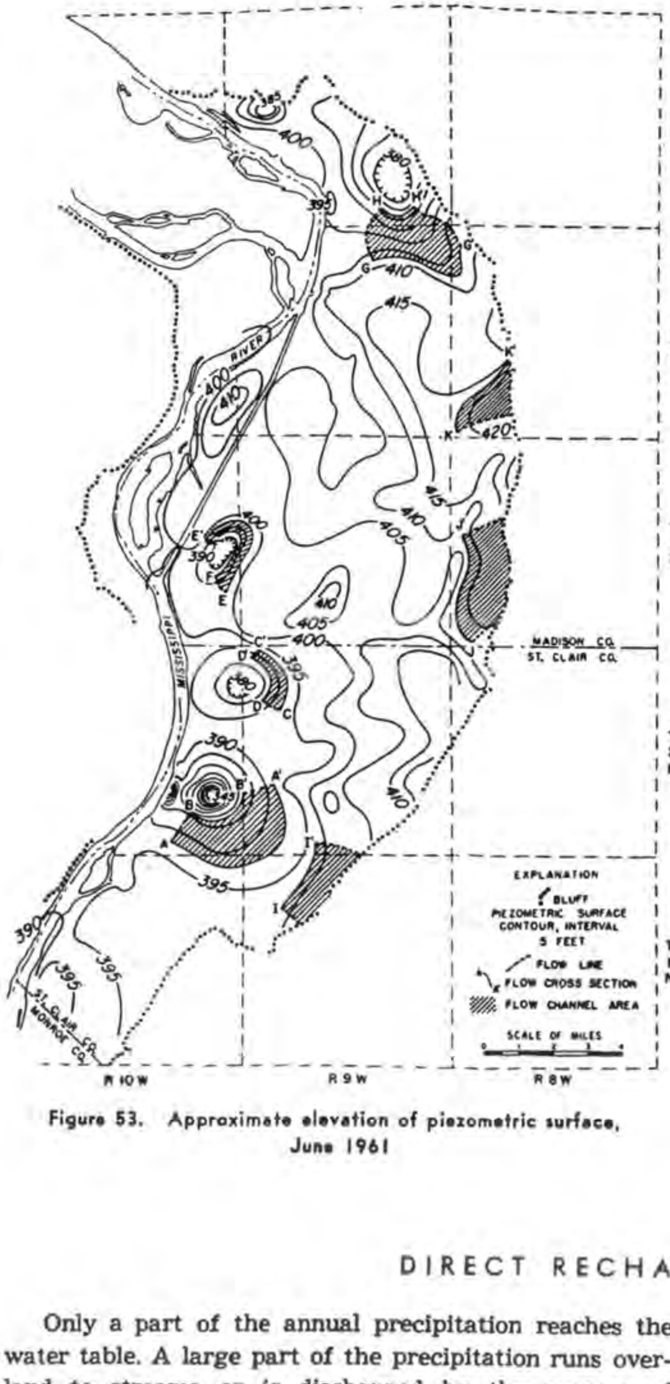


Figure 53. Approximate elevation of piezometric surface, June 1961

Table 24. Lake and Stream Elevations

Gage number	Location of gage	Elevation of measuring point (ft above msl)	Water-surface elevation June 6, 1962 (ft above msl)
2	Highway bridge 2, NW cor, sec 14, T4N, R9W	440.42	414.03
3	Highway bridge 3, NE cor, sec 14, T4N, R9W	441.38	414.09
4	Highway bridge 4, SE cor, sec 12, T4N, R9W	442.95	414.22
1	State Rte 3 bridge, SW cor, sec 5, T2N, R9W	409.80	396.43
2	Sand Prairie Road bridge, Canteen Creek, near center sec 35, T3N, R9W	418.04	400.89
3	Sand Prairie Road bridge, NW cor, sec 35, T3N, R9W	418.55	400.33
4	Hadley bridge, NW cor, sec 19, T3N, R8W	416.40	404.19
5	Black Lane bridge, Canteen Creek, near center sec 36, T3N, R9W	420.80	402.10
	Horseshoe Lake Control Works, NW cor, sec 34, T3N, R9W	403.71	403.64
	Chain of Rocks Canal (upper), SW cor, sec 14, T3N, R10W	(Surface water elevations reported)	407.90
	Chain of Rocks Canal (lower), NW cor, sec 23, T3N, R10W		401.08

Table 25. Mississippi River Stages, June 1962

Gage description	Mississippi River mile number	Water-surface elevation June 8, 1962 (ft above msl)
Lock and Dam No. 26 Alton, Ill. (lower)	202.7	410.6
Hartford, Ill.	196.8	409.4
Chain of Rocks, Mo., pool	190.4	405.5
Tailwater	190.3	404.5
Bissell Point, Mo.	183.3	401.4
St. Louis, Mo.	179.6	399.8
Engineer Depot, Mo.	176.8	398.4

DIRECT RECHARGE TO AQUIFER

Only a part of the annual precipitation reaches the water table. A large part of the precipitation runs overland to streams or is discharged by the process of evapotranspiration before it reaches the aquifer. The amount of precipitation that reaches the zone of saturation depends upon several factors. Among these are the character of the soil and other materials above the water table; the topography; vegetal cover; land use; soil moisture; the depth to the water table; the intensity duration, and seasonal distribution of rainfall; the occurrence of precipitation as rain or snow; and the air temperature.

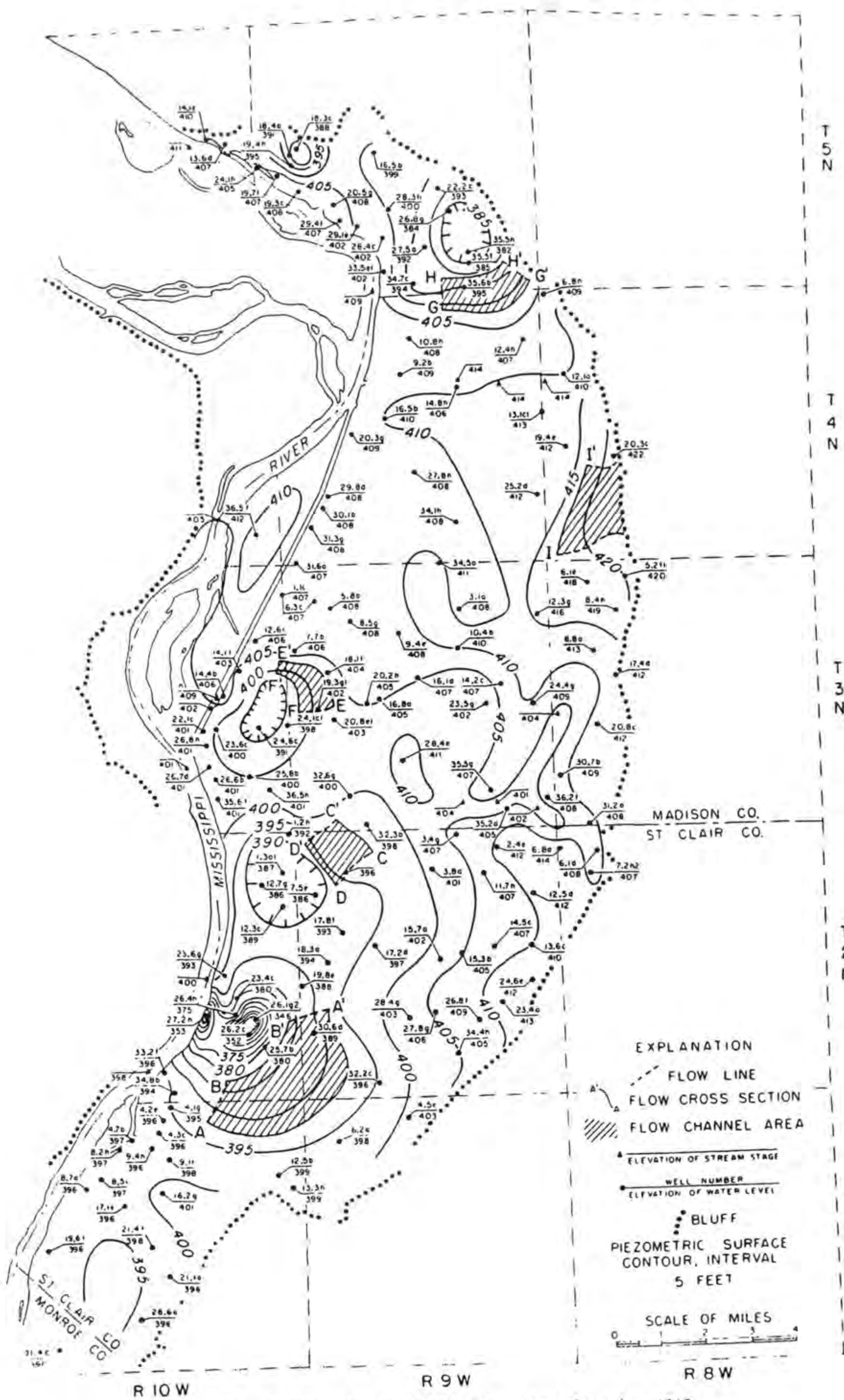


Figure 54. Approximate elevation of piezometric surface, June 1962

Generally ground-water recharge in the East St. Louis area is greatest in spring and early summer months of heavy rainfall and least in the late summer, fall, and winter months. Most recharge occurs during spring months when evapotranspiration is small and soil moisture is maintained at or above field capacity by frequent rains. During summer and fall months evapotranspiration and soil moisture requirements have first priority on precipitation and are so great that little precipitation percolates to the water table except during periods of excessive rainfall.

Recharge directly from precipitation was estimated by flow-net analyses of the piezometric surface in the vicinity of the Wood River, Granite City, National City, and Monsanto area pumping centers. The quantity of water percolating through a given cross section of an aquifer is proportional to the hydraulic gradient (slope of the piezometric surface) and the coefficient of transmissibility, and it can be computed by using the following modified form of the Darcy equation (see Ferris, 1959).

$$Q = TIL \tag{10}$$

where:

Q = discharge through flow cross section, in gpd

T = coefficient of transmissibility, in gpd/ft

I = hydraulic gradient, in ft/mi

L = width of flow cross section, in mi

The rate of recharge directly from precipitation can be estimated on the basis of the difference in discharge of water through successive flow cross sections with the following equation (Walton, 1962):

$$R = [(Q_2 - Q_1) \pm \Delta h_t \, S \, A_t (2.1 \times 10^8)] / A_t \tag{11}$$

where:

R = rate of recharge, in gpd/sq mi

$Q_2 - Q_1$ = difference in discharge of water through successive flow cross sections, in gpd

Δh_t = average rate of water-level decline or rise within area between successive flow cross sections, in fpd

A_t = surface area between successive flow cross sections, in sq mi

S = coefficient of storage of aquifer, fraction

The + sign is used when there is a water-level rise and the − sign is used when there is a water-level decline.

Flow lines were drawn at right angles to the estimated piezometric surface contours for December 1956, June 1961, and June 1962 toward cones of depression in the Wood River, Granite City, National City, and Monsanto areas to delimit the flow channels in figures 52 through 54. The locations of flow channels were so chosen that recharge rates under all types of geologic, hydrologic, and land use conditions could be studied. The discharges through cross sections A—A′, B—B′, C—C′, D—D′, E—E′, F—F′, G—G′, and H—H′ were computed using equation 10 and figures 25 and 52 through 54. Differences in discharge of water through successive flow cross sections were determined. Average rates of water-level declines or rises within flow channel areas were estimated from hydrographs of observation wells. Surface areas of flow channels were obtained from figures 52 through 54. The average coefficient of storage of the coarser deposits was estimated to be 0.20 on the basis of aquifer-test data, and the average coefficient of storage of the finer grained alluvium was estimated to be 0.10 on the basis of studies by Schicht and Walton (1961). The data mentioned above were substituted in equation 11, and recharge rates for each flow channel area were computed.

Recharge rates vary from 299,000 gpd/sq mi in the National City area to 475,000 gpd/sq mi in the Wood River area. The average rate of recharge in the East St. Louis area is 371,000 gpd/sq mi. The East St. Louis area covers about 175 square miles. It is estimated that total recharge directly from precipitation to the East St. Louis area averages about 65 mgd.

The subsurface flow of water from the bluff was estimated by studying the movement of water through flow channels near the foot of the bluff. Flow lines were drawn at right angles to the bluff and the estimated piezometric surface contours for June 1961 and June 1962 to delimit the flow channels shown in figures 53 and 54. The discharge through cross sections I—I′, J—J′, and K—K′ were computed using equation 10 and figures 25, 53, and 54. Average rates of water-level declines or rises within flow channel areas were estimated from hydrographs of observation wells. The average rates of changes in storage within flow channel areas were computed as the products of water-level changes, storage coefficients, and flow channel areas. Recharge directly from precipitation within flow channel areas was estimated as the products of the average recharge rate (371,000 gpd/sq mi) and flow channel areas. Recharge and changes in storage within flow channel areas were subtracted from the discharges through cross sections I—I′, J—J′, and K—K′ to compute rates of subsurface flow of water from the bluff. The average rate of subsurface flow of water from the bluff is 329,000 gpd/mi. The length of the bluff forming the eastern boundary of the East St. Louis area is 39 miles. Thus, the total rate of subsurface flow of water from the bluffs is about 12.8 mgd.

RECHARGE FROM INDUCED INFILTRATION

The lowering of water levels in the Alton, Wood River, National City, and Monsanto areas that has accompanied withdrawals of ground water in these areas has established hydraulic gradients from the Mississippi River towards these pumping centers. In addition, lowering of water levels in the Granite City area has established a hydraulic gradient from the Chain of Rocks Canal towards the Granite City pumping center. Thus, ground-water levels are below the surface of the river and canal at places, and appreciable quantities of water percolate through the beds of the river and canal into the aquifer by the process of induced infiltration.

The volume of water percolating through the beds of the river and canal into the aquifer during 1961 was estimated by subtracting the volume of water recharged to the aquifer within areas of diversion directly from precipitation and subsurface flow from the bluff from the total volume of water pumped. In 1961 cones of depression were relatively stable and changes in storage within the aquifer during the year were very small. As shown in table 26 about 48.2 mgd or 50.0 percent of the total

Table 26. Recharge by Source During 1961

Pumping center	Total pumpage (mgd)	Length of bluff within area of diversion (mi)	Recharge from bluff (mgd)	Area of diversion (sq mi)	Recharge from precipitation (mgd)	Recharge by induced infiltration (mgd)
Alton area	12.30	3.4	1.12	2.7	1.00	10.18
Wood River area	24.30	7.9	2.60	19.5	7.24	14.46
Poag area	1.20	neg	neg	3.2	1.20	0
Granite City area	8.80	0	0	20.6	7.65	1.15
Troy area	0.40	neg	neg	1.1	0.40	0
National City area	10.80	0	0	18.7	6.94	3.86
Fairmont City area	4.40	0	0	11.8	4.40	0
Caseyville area	2.40	2.9	0.95	3.9	1.44	0
Glen Carbon area	0.30	neg	neg	0.8	0.30	0
Monsanto area	31.90	2.3	0.76	34.0	12.61	18.53
Total	96.80		5.43		43.18	48.18

pumpage (96.8 mgd) was derived from induced infiltration of surface water in the Mississippi River. The piezometric surface map in figure 54 was used to delimit areas of diversion and lengths of bluff within areas of diversion. Recharge directly from precipitation was estimated as the products of areas of diversion and the average recharge rate (371,000 gpd/sq mi). Subsurface flow from the bluff was estimated as the products of lengths of bluff within areas of diversion and the average rate of subsurface flow (329,000 gpd/mi).

The amount of induced infiltration is dependent largely upon the infiltration rate of the river bed, the river-bed area of infiltration, the position of the water table, and the hydraulic properties of the aquifer.

Infiltration Rates of River Bed

The infiltration rate of the Mississippi River bed was determined with aquifer-test data. Methods of analysis of aquifer-test data affected by stream recharge were described by Rorabaugh (1956), and Hantush (1959). In addition, Walton (1963) introduced a method for determining the infiltration rate of a stream bed by aquifer-test analysis.

If the hydraulic properties of the aquifer and the distance a are known, the percentage of pumped water being diverted from a stream can be computed with the following equation derived by Theis (1941):

$$P_r = 2/\pi \int_0^{\pi/2} \exp(-f \sec^2 u)\, du \qquad (12)$$

where:

$u = \tan^{-1}(r_r/a)$
$f = 1.87a^2S/Tt$
P_r = percentage of pumped water being diverted from the stream
T = coefficient of transmissibility, in gpd/ft
S = coefficient of storage, fraction
a = distance from pumped well to recharge boundary, in ft
t = time after pumping started, in days
r_r = distance along recharge boundary measured from the perpendicular joining the real and image wells, in ft

Figure 55 gives values of P_r for various values of f and shows, therefore, the percentage of pumped water being diverted from the stream. The amount of recharge by induced infiltration is then given by the following equation:

$$Q_r = QP_r/100 \qquad (13)$$

where:

Q_r = amount of induced infiltration, in gpm
Q = discharge of pumped well, in gpm

Values of drawdown at several points within the stream bed equidistant upstream and downstream from the pumped well and between the line of recharge and the river's edge are computed, taking into consideration the effects of the image well associated with the line of recharge and the pumped well, with the following equations:

$$s = s_p - s_i \qquad (14)$$

$$s_p = 114.6QW(u_p)/T \qquad (15)$$

$$s_i = 114.6QW(u_i)/T \quad (16)$$

$$u_p = 2693r_p^2S/Tt \quad (17)$$

$$u_i = 2693r_i^2S/Tt \quad (18)$$

where:

s = drawdown at observation point, in ft
s_p = drawdown due to pumped well, in ft
s_i = buildup due to image well, in ft
Q = discharge of pumped well, in gpm
T = coefficient of transmissibility, in gpd/ft
S = coefficient of storage, fraction
r_p = distance from observation point to pumped well, in ft
r_i = distance from observation point to image well, in ft
t = time after pumping started, in min

The reach of the streambed, L_r, within the area of influence of pumping is determined by noting the location of the points upstream and downstream where drawdown is negligible (say $\leqq 0.01$). The area of induced infiltration, A_r, is then the product of L_r and the average distance between the river's edge and the recharge boundary.

The infiltration rate of the stream bed per unit area can be computed with the following equation:

$$I_a = 6.3\times10^7Q_r/A_r \quad (19)$$

where:

I_a = average infiltration rate of stream bed, in gallons per day per acre (gpd/acre)
Q_r = amount of induced infiltration, in gpm
A_r = stream bed area of infiltration, in sq ft

Rough approximations of the average head loss, s_r, due to the vertical percolation of water through the stream bed can be determined by averaging drawdowns computed at many points within the area of infiltration. Values of drawdown within the stream-bed area of infiltration are computed, taking into consideration the pumped well and the image well associated with induced infiltration, with equations 14 through 18.

The average infiltration rate of the stream bed per unit area per foot of head loss can be estimated by use of the following equation:

$$I_h=I_a/s_r \quad (20)$$

where:

I_h = average infiltration rate of stream bed, in gallons per day per acre of stream bed per foot of head loss (gpd/acre/ft)
I_a = average infiltration rate of stream bed, in gpd/acre
s_r = average head loss within the stream bed area of infiltration, in ft

The infiltration rate of the Mississippi River bed at three sites was determined from aquifer-test data. The sites are just south of the confluence of Wood River and

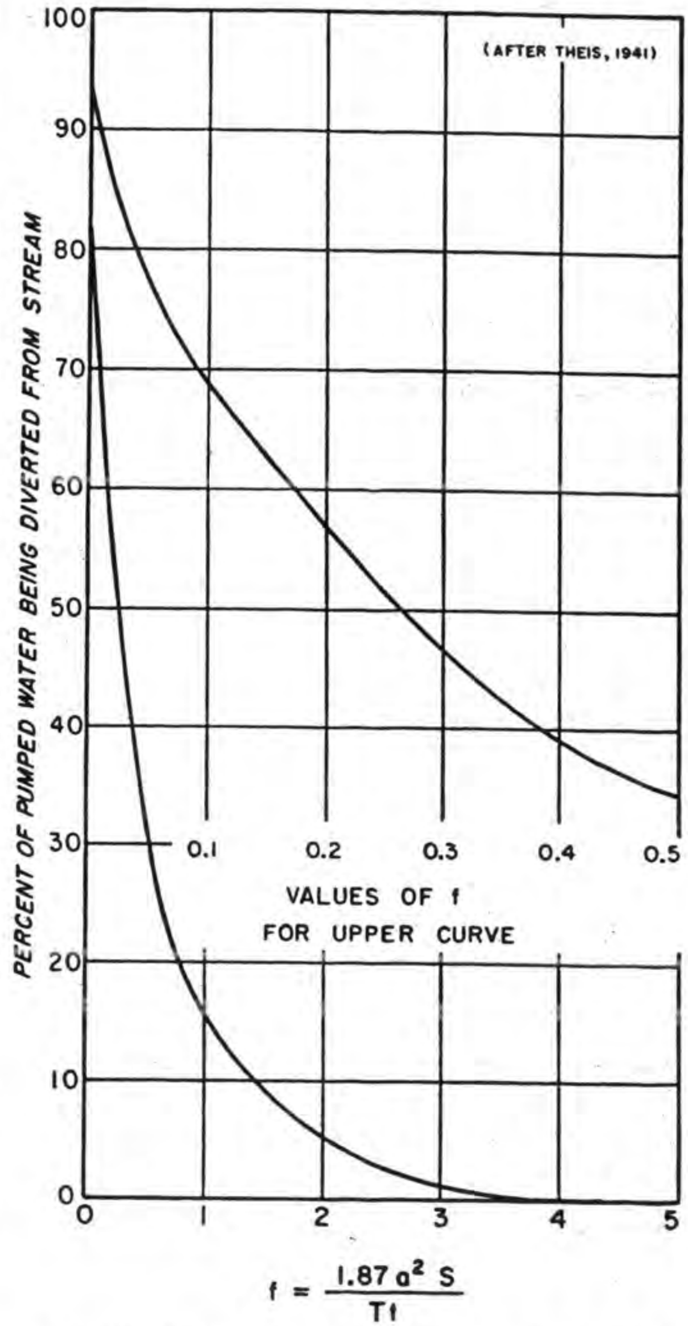


Figure 55. Graph showing the relationship between percent of pumped water being diverted from a stream and the factor 'f'

the Mississippi River, west of Wood River, and west of Monsanto. A summary of the results of aquifer tests and computed infiltration rates are given in table 27. The infiltration rate near the confluence of Wood River and the Mississippi River at a river temperature of 33F was estimated to be 305,000 gpd/acre/ft; the infiltration rate west of the city of Wood River was estimated to be 36,300 gpd/acre/ft; and the infiltration rate west of Monsanto at a river temperature of 83F was estimated to be 91,200 gpd/acre/ft.

Infiltration rates per foot of head loss vary with the temperature of the river water. Average monthly infiltra-

Table 27. Results of Aquifer Tests Affected by Induced Infiltration

Owner	Location	Date of test	Duration of test (days)	Pumping rate (gpm)	Hydraulic properties T (gpd/ft)	P (gpd/sq ft)	S (fraction)	I_a (gpd/acre)	s_r (ft)	I_h (gpd/acre/ft)	River temperature (°F)
Olin Mathieson Chemical Corp.	Madison Cty. T5N, R9W sec 19	May 29-Jun 1, 1956; Feb 13-17, 1959	3 4	760 7000	100,000	1100	0.1	418,000	1.37	305,000	33
Shell Oil Co.	Madison Cty. T5N, R9W sec 33	Mar 3-6, 1952	3	510	190,000	1900	0.002	9,800	0.27	36,300	38
Monsanto Chemical Corp.	St. Clair Cty. T2N, R10W sec 27	Aug 4-8, 1952	4	1100	210,000	2800	0.08	15,500	0.17	91,200	83

tion rates (tables 28 and 29) were computed on the basis of average monthly river temperatures, figure 56, and the following equation:

$$I_t = I_h(\mu_a/\mu_t) \tag{21}$$

where:

I_t = average infiltration rate of river bed for a particular surface water temperature, in gpd/acre/ft

I_h = average infiltration rate of river bed determined from aquifer-test results, in gpd/acre/ft

μ_a = coefficient of viscosity at temperature of surface water during aquifer test, in centimeter-gram-seconds (cgs) units

μ_t = coefficient of viscosity at a particular temperature of surface water, in cgs units

Table 28. Average Monthly Infiltration Rates of Mississippi River Bed near Alton and Wood River

Month	Average river temperature at Alton 1940-1949 (°F)	Infiltration rate of river bed (gpd/acre/ft): West of Wood River	Near confluence of Wood and Mississippi Rivers
January	34	33,800	308,000
February	34	33,800	308,000
March	41	38,500	350,000
April	54	47,600	436,000
May	64	54,600	497,000
June	74	63,100	574,000
July	81	69,200	636,000
August	82	70,000	643,000
September	75	63,700	571,000
October	63	54,600	493,000
November	50	44,600	406,000
December	38	36,300	330,000

River-Bed Areas of Infiltration to Well Fields

Four well fields in the East St. Louis area are located close to the Mississippi River and derive most of their recharge from the induced infiltration of surface water. The well fields are south of Alton in the Duck ake area, near the confluence of the Wood River and ıe Mississippi River, west of Wood River, and west of Monsanto as shown in figure 57.

One well field consisting of a collector well and two artificial pack wells is owned by the Shell Oil Refinery and is located about 100 feet east of the Mississippi River west of Wood River in sec 33, T5N, R9W. The design capacity of the well field is 5000 gpm or 7.2 mgd.

The position of the recharge boundary and the area of infiltration for the design capacity were determined by the process of trial and error. Several positions of the recharge boundary were assumed, and drawdown

Table 29. Average Monthly Infiltration Rates of Mississippi River Bed near Monsanto

Month	Average river temperature at East St. Louis 1940-1949 (°F)	Infiltration rate of river bed (gpd/acre/ft)
January	38	47,600
February	38	47,600
March	43	49,500
April	55	62,200
May	66	71,500
June	76	83,100
July	82	90,100
August	83	91,200
September	77	84,000
October	65	72,000
November	53	61,400
December	41	49,300

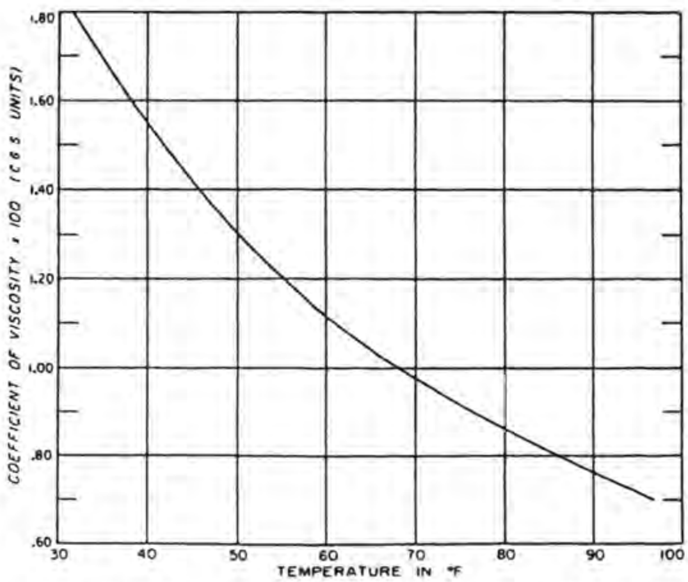


Figure 56. Graph showing relationship between coefficient of viscosity and temperature

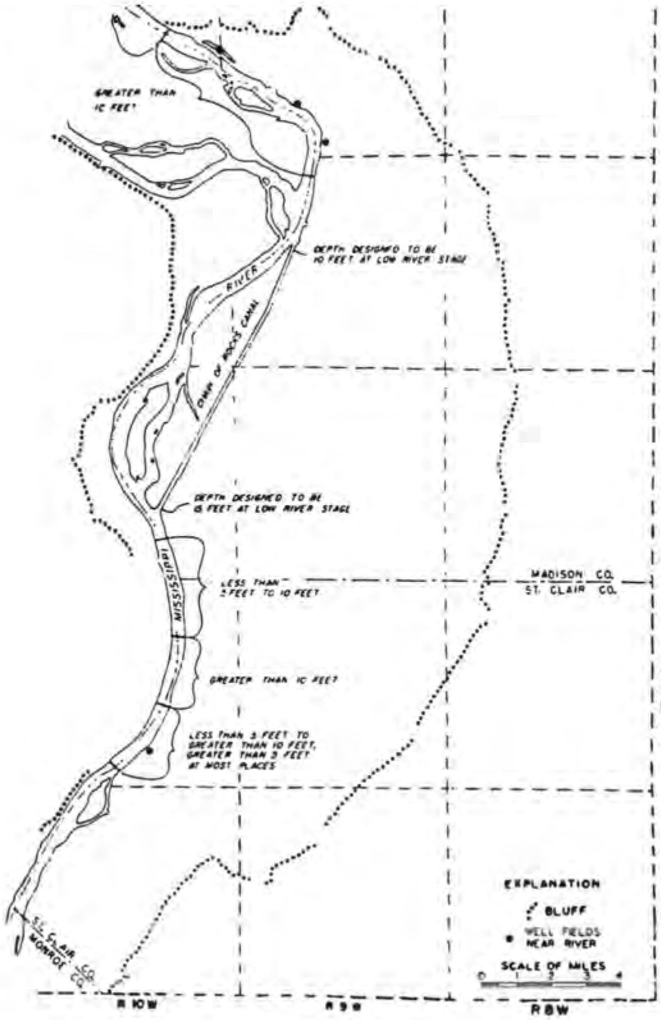


Figure 57. Estimated depths of Mississippi River and locations of well fields near river

beneath the river bed and the river-bed areas of infiltration were computed with equations 14 through 18. Values of R_i were then computed with equation 22 keeping in mind that s_r is either the average head loss within the river-bed area of infiltration or the average depth of water in the river, depending upon the drawdown beneath the river bed.

$$R_i = I_t\, s_r\, A_r \tag{22}$$

where:

R_i = potential recharge by induced infiltration, in gpd
I_t = average infiltration rate of river bed for a particular surface water temperature, in gpd/acre/ft
s_r = average head loss within river bed area of infiltration or average depth of water in river for a particular river stage, depending upon the position of the water table, in ft
A_r = river bed area of infiltration, in acres

The position of the recharge boundary and the river-bed area of infiltration which resulted in R_i balancing the design capacity were judged to be correct. The recharge boundary for the design capacity is located at a distance of 900 feet from the well field and the river-bed area of infiltration is 175 acres, as shown in figure 58.

The results of an aquifer test, made at a low pumping rate at the site of the well field, indicated a distance of 500 feet from the well field to the recharge boundary. Thus, the aquifer test at a low pumping rate indicated a certain position of the recharge boundary and a river-bed area of infiltration which were not valid for a higher pumping rate. At higher pumping rates water is withdrawn at a rate in excess of the ability of the river-bed to transmit it, and as a result the water table declines below portions of the river-bed. In such a case the recharge boundary moves away from the pumped wells as maximum infiltration occurs in the reach of the river in the immediate vicinity of the well field, the cone of depression spreads upstream and downstream, and the river-bed area of infiltration increases. Drawdowns in wells at higher pumping rates based on the position of the recharge boundary as determined from the aquifer-test data are much less than drawdowns based on the position of the recharge boundary as determined by trial and error with equation 22. Thus, the position of the recharge boundary determined from aquifer-test data cannot always be used to compute the potential yield of well fields that depend primarily upon induced infiltration of surface water as a source of recharge.

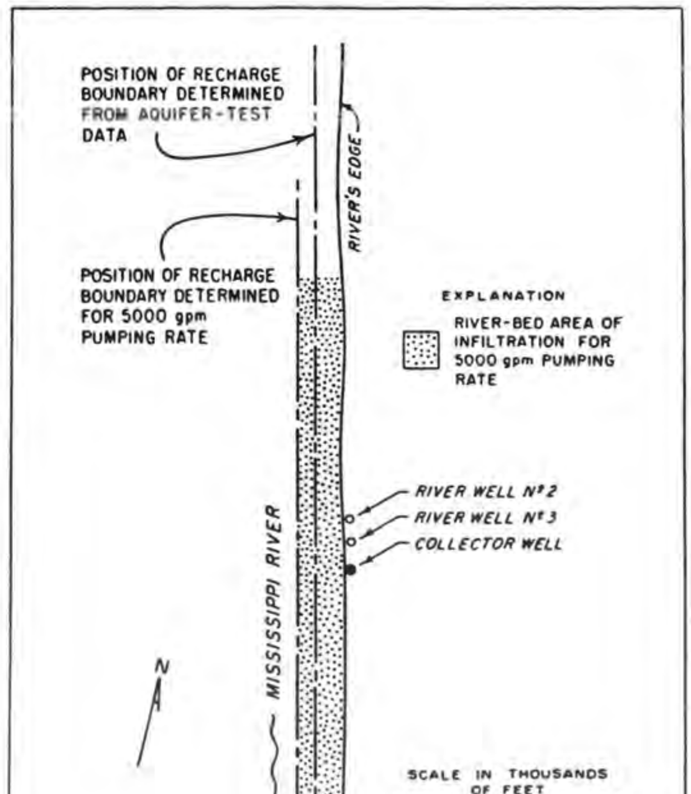


Figure 58. River-bed area of infiltration for Shell Oil Refinery well field

Potential recharge by the induced infiltration of sur- e water can be estimated on the basis of the infiltra- n rates in table 30, river depth records, water-level data, and river temperature data. Infiltration is directly proportional to the drawdown immediately below the stream bed and is at a maximum when the water table is immediately below the river bed. Under maximum infiltration conditions the average head loss within the river-bed area of infiltration is the average depth of water in the river for a particular river stage. Provided the water table remains belo amounts of induced infiltr dry periods when streamflo surface water are low. Prof be used to determine the a river. Potential recharge b determined by substituting

Table 30. Infiltration Rates of Stream Beds Determined from Aquifer-Test Data in Illinois, Indiana, and Ohio

Location of aquifer-test site	Infiltration rate (gpd/acre/ft)	Surface water temperature (°F)	Infiltration rate at 40 F (gpd/acre/ft)
Along Mad River about miles northwest of pringfield, Ohio*	1,000,000	39	1,010,000
Along Miami River 14 miles northwest of Cincinnati, Ohio*	168,000	82	91,100
Along White River immediately upstream from the confluence of White River and Killbuck Creek at Anderson, Indiana*	216,000	54	275,000
Along Sandy Creek 12 miles south of Canton, Ohio*	720,000	69	414,000
Along White River 1 mile west of Anderson, Indiana, ½ mile below sewage treatment plant*	39,800	35	43,600
Along Mississippi River near confluence of Wood River and Mississippi River above confluence of Mississippi and Missouri Rivers	305,000	33	344,000
Along Mississippi River west of the city of Wood River above confluence of Mississippi and Missouri Rivers	36,300	38	37,500
long Mississippi River vest of Monsanto below confluence of Mississippi and Missouri Rivers	91,200	83	48,300

*After Walton (1963)

The average depth of water in the Mississippi River between the Illinois shore and a line 500 feet offshore was estimated from Mississippi River soundings made by the U.S. Corps of Engineers and low river stages during 1956 and 1957. The average depth of water exceeds 10 feet in places where the navigation channel is near the Illinois side, in the vicinity of Alton and Wood River, and along a small reach of the river near East St. Louis. The depth of water in the Chain of Rocks Canal is designed to be 10 feet or greater at low river stages. Estimated average depths of water in the river at low river stages are shown in figure 57.

A summary of the infiltration rates computed with aquifer-test data for the East St. Louis area is given in table 30. Infiltration rates of stream beds in Ohio and Indiana (Walton, 1963) are also listed. Infiltration rates in table 30 were adjusted to a river temperature of 40F. A comparison of the adjusted infiltration rates with infiltration rate data for slow and rapid sand filters (Fair and Geyer, 1954) indicates that all stream bed infiltration rates fall into the clogged slow sand filter category.

The least permeable reach of river bed in the East St. Louis area is west of Wood River above the confluence of the Mississippi and Missouri Rivers. The infiltration rate along this reach and the infiltration rate of the reach of river bed west of Monsanto below the confluence of the Mississippi and Missouri Rivers are low and in the same range as the infiltration rate for the White River west of Anderson, Indiana, below a sewage treatment plant. Walton (1963) states that the infiltration rate of the White River site is probably low largely because of the clogging effects of sewage.

The highest infiltration rate in the East St. Louis area was computed for the reach of river bed near the confluence of the Wood and Mississippi Rivers above the confluence of the Mississippi and Missouri Rivers. The Missouri River generally carries a greater sediment load than the Mississippi River; thus it would be expected that the average infiltration rate above the Missouri River would be greater than the average infiltration rate below it.

The infiltration rate of the Mississippi River bed west of the city of Wood River ranges from 33,800 gpd/acre/ft at an average river temperature of 34F in January and February to 70,000 gpd/acre/ft in August when the average river temperature is 82F. The infiltration rate of the river bed near the confluence of the Wood and the Mississippi Rivers ranges from 308,000 gpd/acre/ft in January and February to 643,000 gpd/acre/ft in August. West of Monsanto the infiltration rate of the river bed varies from 47,600 gpd/acre/ft at an average river temperature of 38F in January and February to 91,200 gpd/acre/ft at an average river temperature of 83F in August.

ELECTRIC ANALOG COMPUTER

An electric analog computer (see Walton and Prickett, 1963) for the East St. Louis area was constructed so that the consequences of further development of the aquifer could be forecast, the practical sustained yield of existing pumping centers could be evaluated, and the potential yield of the aquifer with a selected scheme of development could be appraised. The electric analog computer consists of an analog model and excitation-response apparatus, i.e., waveform generator, pulse generator, and oscilloscope.

The analog model is a regular array of resistors and capacitors and is a scaled down version of the aquifer. Resistors are inversely proportional to the coefficients of transmissibility of the aquifer, and capacitors store electrostatic energy in a manner analogous to the storage of water in the aquifer. Hydrogeologic maps and data presented earlier in this report describing the following factors were used in constructing the analog model: 1) coefficient of transmissibility of the aquifer, 2) coefficient of storage of the aquifer, 3) areal extent of the aquifer, 4) saturated thickness of the aquifer, and 5) location, extent, and nature of aquifer boundaries. All nonhomogeneous and irregular hydrogeologic conditions were incorporated in the analog model.

Questions pertaining to the utilization of groundwater resources of the East St. Louis area require that pumping be related to water-level change with reference to time and space. Changes in water levels due to the withdrawal of water from the aquifer must be determined. Excitation-response apparatus force electric energy in the proper time phase into the analog model and measure energy levels within the energy-dissipative resistor-capacitor network. Oscilloscope traces, i.e., time-voltage graphs, are analogous to time-drawdown graphs that would result after a step function-type change in withdrawal of water. A catalog of time-voltage graphs provides data for construction of a series of water-level change maps. Thus, the electric analog computer provides a means of relating cause and effect relationships for the aquifer. A schematic diagram of the electric analog computer is shown in figure 59.

Analog Model

The analog model for simulating the aquifer in the East St. Louis area was patterned after analog models developed by H. E. Skibitzke, mathematician, U.S. Geological Survey, Phoenix, Arizona. The analog model consists of a regular array of 2800 resistors and 1350 capacitors. The analog model was constructed with a piece of 1/8-inch pegboard perforated with holes on a 1-inch square pattern approximately 2 x 5 feet corresponding to the dimensions of the topographic map of the area (7.5 minute quadrangle maps). Aluminum angles (1 x 1 inch) were attached along the four edges of the pegboard with metal screws to enable setting the model on a table or against a wall without disturbing capacitors of the analog model installed on the underneath side of the pegboard. Coefficient of transmissibility contours were transferred from figure 25 to topographic maps of the area which were in turn pasted on the pegboard. No. 3 brass laquered shoe eyelets were inserted in the holes of the pegboard to provide terminals for resistors and capacitors. Four resistors and a capacitor were connected to each interior terminal; the capacitor was secured to a ground wire connection of the electrical system. Two or three resistors and a capacitor were connected to boundary terminals, depending upon the geometry of the boundary. The model is bounded on the west by a recharge boundary, the Mississippi River and the Chain of Rocks Canal; the portion of the network along the recharge boundary was terminated in a short circuit. The recharge boundary of the network was adjusted in a step fashion to approximate the actual boundary of the aquifer. The model is bounded on the north, east, and southeast, by bluffs through which there is a small amount of subsurface flow. Resistors large in magnitude which simulate small amounts of subsurface flow through the bluff were connected to terminals along the north, east, and southeast boundaries of the analog model and to the ground connection of the electrical system. The model was terminated south of Dupo. A termination strip was constructed to extend the aquifer 5 miles south of Dupo (see Karplus, 1958).

Because the aquifer is a continuous phenomena while the resistor-capacitor network consists of many discrete branches, the network is only an approximation of a true analog. However, it can be shown mathematically that if the mesh size of the network is small in comparison with the size of the aquifer, the behavior of the network describes very closely the response of the aquifer to pumping.

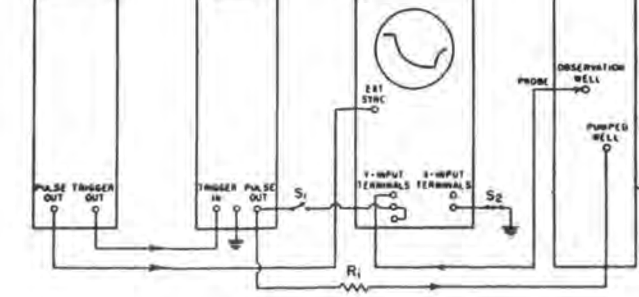


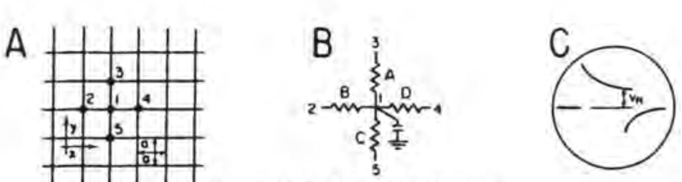
Figure 59. Schematic diagram of electric analog computer

The model was developed on the premise that ground-water flow in the East St. Louis area is two-dimensional. The finite-difference form of the partial differential equation (Jacob, 1950) governing the nonsteady state two-dimensional flow of ground-water is (see Stallman, 1956):

$$T\ (\Sigma_2^5\ h_i - 4h_1)\ =\ a^2\ S\ (\partial h/\partial t) \qquad (23)$$

where:

h_1 = head at node 1 (see figure 60A; the aquifer is subdivided into small squares of equal area, the intersections of grid lines are called nodes); h_i (i = 2, 3, 4, and 5) = heads at nodes 2 to 5; a = width of grid interval; T = coefficient of transmissibility; and S = coefficient of storage.



(FROM WALTON & PRICKETT, 1963)

Figure 60. Finite-difference grid (A), resistor-capacitor net (B), and pumping rate oscilloscope trace (C)

Consider a resistor-capacitor network with a square pattern as shown in figure 60A and network junctions at nodes as defined in figure 60B. The junctions consist of four resistors of equal value and one capacitor connected to a common terminal; the capacitor is also connected to ground. The relation of electrical potentials in the vicinity of the junction, according to Kirchhoff's current law, can be expressed by the following equation (see Millman and Seely, 1941; and Skibitske, 1961):

$$1/R\ (\Sigma_2^5\ V_i - 4V_1)\ =\ C\ (\partial V/\partial t) \qquad (24)$$

where:

$V_{1\text{-}5}$ = electrical potential at ends of resistors; $R_{A\text{-}D}$ = resistance; and C = capacitance; V_i (i=2, 3, 4, and 5) = electrical potential at ends of resistors A–D.

Comparison of equations 23 and 24 shows that the finite-difference equation governing the nonsteady state two-dimensional flow of ground water in an infinite aquifer is of the same form as the equation governing the flow of electrical current in a resistor-capacitor network. For every term in equation 23 there is a corresponding term of the same order of differentiation in equation 24.

The analogy between electrical and aquifer systems is apparent. The hydraulic heads, h, are analogous to electrical potentials, V. The coefficient of transmissibility, T, is analogous to the reciprocal of the electrical resistance, $1/R$. The product of the coefficient of storage, S, and a^2 is analogous to the electrical capacitance, C.

Continuing the comparison, water moves in an aquifer just as charges move in an electrical circuit. The quantity of water is reckoned in gallons while the charge is in coulombs. The rate of flow of water past any point in the aquifer is expressed in gallons per day while the flow of electricity is in coulombs per second or amperes. The hydraulic head loss between two points in an aquifer is expressed in feet while the potential drop across a part of the electrical circuit is in volts.

Thus, there are four units which are analogous; there is necessarily a scale factor connecting each unit in one system to the analogous unit in the other system. Knowing the four scale factors the hydrologist is able to relate electrical units associated with the analog model to hydraulic units associated with an aquifer. The four scale factors, K_1, K_2, K_3, and K_4, were defined by Bermes (1960) as follows:

$$q = K_1\Omega \qquad (25)$$

$$h = K_2V \qquad (26)$$

$$Q = K_3I \qquad (27)$$

$$t_d = K_4t_s \qquad (28)$$

where:

q = gallons; Ω = coulombs; Q = gallons per day; I = amperes; h = feet; V = volts; t_d = days; t_s = seconds; K_1 = gal/coulomb; K_2 = feet/volt; K_3 = gal/day/ampere; and K_4 = days/sec.

The relation between scale factors K_1, K_3, and K_4 is expressed by the following equation (Bermes, 1960):

$$(K_3K_4)/K_1 = 1 \qquad (29)$$

The analogy between Ohm's law and Darcy's law is established by the fact that the coefficient of transmissibility is analogous to the reciprocal of the electric resistance. Substitution of these laws in equation 27 results in the following equation which may be used to determine the values of the resistors of the interior portions of the analog model (see Bermes, 1960):

$$R = K_3/(K_2T) \qquad (30)$$

where:

R = resistance, in ohms; and T = coefficient of transmissibility, in gpd/ft.

The following equation (see Bermes, 1960), which may be used to determine the values of the capacitors of the interior portions of the analog model, may be derived by taking into consideration the definitions of the coefficient of storage and capacitance and the analogy between (a^2S) and C.

$$C = 7.48\ a^2\ S\ (K_2/K_1) \qquad (31)$$

where:

C = capacitance, in farads; a = network spacing, in feet; and S = coefficient of storage, fraction.

A network spacing of 1 inch equals 2000 feet was selected to minimize the errors due to finite-difference approximation. Equations given by Karplus (1958) suggest that the selected network spacing is adequate.

By the process of trial and error, scale factors were chosen so that readily available and inexpensive resistors and capacitors and existing excitation-response apparatus could be used.

Selected analog scale factors are given below:

$K_1 = 1.826 \times 10^{15}$ gallons/coulomb

$K_2 = 1$ ft/volt

$K_3 = 1 \times 10^{10}$ gal/day/amp

$K_4 = 1.826 \times 10^5$ days/sec

A maximum pumping period, t_d, of 5 years was chosen, which is a sufficient period for water levels to stabilize under the influence of recharge from the Mississippi River. According to equation 28, with a $K_4 = 1.826 \times 10^5$ days/sec and when $t_d = 5$ years, the pulse duration, t_s, is equal to 10^{-2} seconds. The pulse generator has a maximum pulse duration of 10^{-2} seconds. A scale factor K_2 of 1 ft/volt was selected for ease in reading the oscilloscope graph.

A generalization of equations 23 and 24 permits accounting for variations in space of the coefficients of transmissibility and storage by varying resistors and capacitors. Fixed carbon resistors with tolerances of $\pm$ 10 percent and ceramic capacitors with tolerances of $\pm$ 10 percent were used in constructing the analog model.

Values of resistors were computed from equation 30 using data on the coefficient of transmissibility given in figure 25. Values of resistors in the internal parts of the model range in magnitude from 470,000 ohms near the bluff where T is about 20,000 gpd/ft to 33,000 ohms near Monsanto where T is about 330,000 gpd/ft. Resistors are greatest in magnitude, 2,200,000 ohms, along the valley wall where the coefficient of transmissibility is about 5000 gpd/ft.

Values of the capacitors of the interior portions of the model were computed from equation 31 to be 2500 micro-micro farads. The long-term coefficient of storage substituted in equation 31 was 0.15.

Excitation-Response Apparatus

The excitation-response apparatus consists of three major parts as shown in figure 60: a waveform generator, a pulse generator, and an oscilloscope. The waveform generator which produces sawtooth pulses is connected to the trigger circuits of the pulse generator and oscilloscope, thereby controlling the repetition rate of computation and synchronizing the oscilloscope's horizontal sweep and the output of the pulse generator. The pulse generator, which produces rectangular pulses of various duration and amplitude upon command from the waveform generator, is coupled to that junction in the analog model representing the pumped well. The oscilloscope is connected to junctions of the analog model where it is desired to determine the response of the analog model to excitation. An electron beam is swept across the cathode ray tube of the oscilloscope providing a time-voltage graph which is analogous to the time-drawdown graph for an observation well. The waveform generator sends a positive pulse to the oscilloscope to start its horizontal sweep; at the same time, it sends a negative sawtooth waveform to the pulse generator. At a point along the sawtooth waveform the pulse generator is triggered to produce a negative rectangular pulse. The duration of this pulse is analogous to the pumping period, t_d, and the amplitude is analogous to the pumping rate, Q. This pulse is sensed by the oscilloscope as a function of the analog model components, boundary conditions, and node position of the junction connected to the oscilloscope. Thus, the oscilloscope trace is analogous to the water-level fluctuation that would result after a step function-type pumpage change of known duration and amplitude. To provide data independent of the pulse repetition rate, the interval between pulses is kept several times the longest time constant in the analog model. The time constant is the product of the capacitance at a point and the resistance in its discharge path.

A means of computing the pumping rate is incorporated in the circuit between the pulse generator and the analog model by the small resistor, R_i, in series, shown in figure 59. Substitution of Ohm's law in equation 27 results in the following equation which may be used to compute the pumping rate:

$$Q = (V_R / 1.44 \times 10^3 R_i)\, K_3 \qquad (32)$$

where:

Q = pumping rate, in gpm; V_R = voltage drop across the resistor R_i, in volts; and R_i = calibrated resistance, in ohms.

The voltage drop across the calibrated resistor is measured with the oscilloscope. Switches S_1 and S_2 are closed and opened, respectively, and the oscilloscope is connected to the pumped well junction. The waveform in figure 60C appears on the cathode ray tube; the vertical distance as shown is the desired voltage drop, V_R.

The switches S_1 and S_2 are returned to their original positions. The oscilloscope is then connected to all junctions of the analog model representing observation wells. The screen of the oscilloscope is accurately calibrated so that voltage and time may be used on the vertical and horizontal axis, respectively. The time is in seconds; the value of each horizontal division on the screen is determined by noting the duration of the rectangular pulse and the number of divisions covered by the time-voltage trace for a junction adjacent to the pumped well. The time-voltage graphs obtained from the oscilloscope can be converted into time-drawdown graphs with equations 26 and 28 which relate electrical units to hydraulic units. A catalog of time-drawdown graphs provides data for the construction of a series of water-level change contour maps. Thus, water-level changes are described everywhere in the aquifer for any desired pumping period. The pulse generator can be coupled to many junctions, and a variety of pumping conditions can be studied.

The effects of complex pumpage changes on water levels may be determined by approximating the pumpage graph by a group of step functions and analyzing the effect of each step function separately. The total water-level change, based on the superposition theorem, is obtained by summation of individual step-function water-level changes.

The pulse generator has a maximum output of 50 volts and 20 milliamperes; the pulse generator and oscilloscope have rise times less than 1 microsecond and waveform durations from less than 10 microseconds to 100 milliseconds. The performance specifications of the waveform generator, pulse generator, and oscilloscope are compatible with the following desired criteria for analog computers: low power requirements, respective calculation at variable rates, and fast computing speeds.

Accuracy and Reliability of Computer

The accuracy and reliability of the electric analog computer were assessed by a study of records of past pumpage and water levels. Water-level declines and piezometric surface maps obtained with the electric analog computer were compared with actual water-level declines and piezometric surface maps. The piezometric surface map for December 1956 (see figure 61A) was used to appraise the accuracy and reliability of the electric analog computer. The effects of the prolonged drought (1952-1956) on water levels are reflected in the piezometric surface. Hydrographs of observation wells

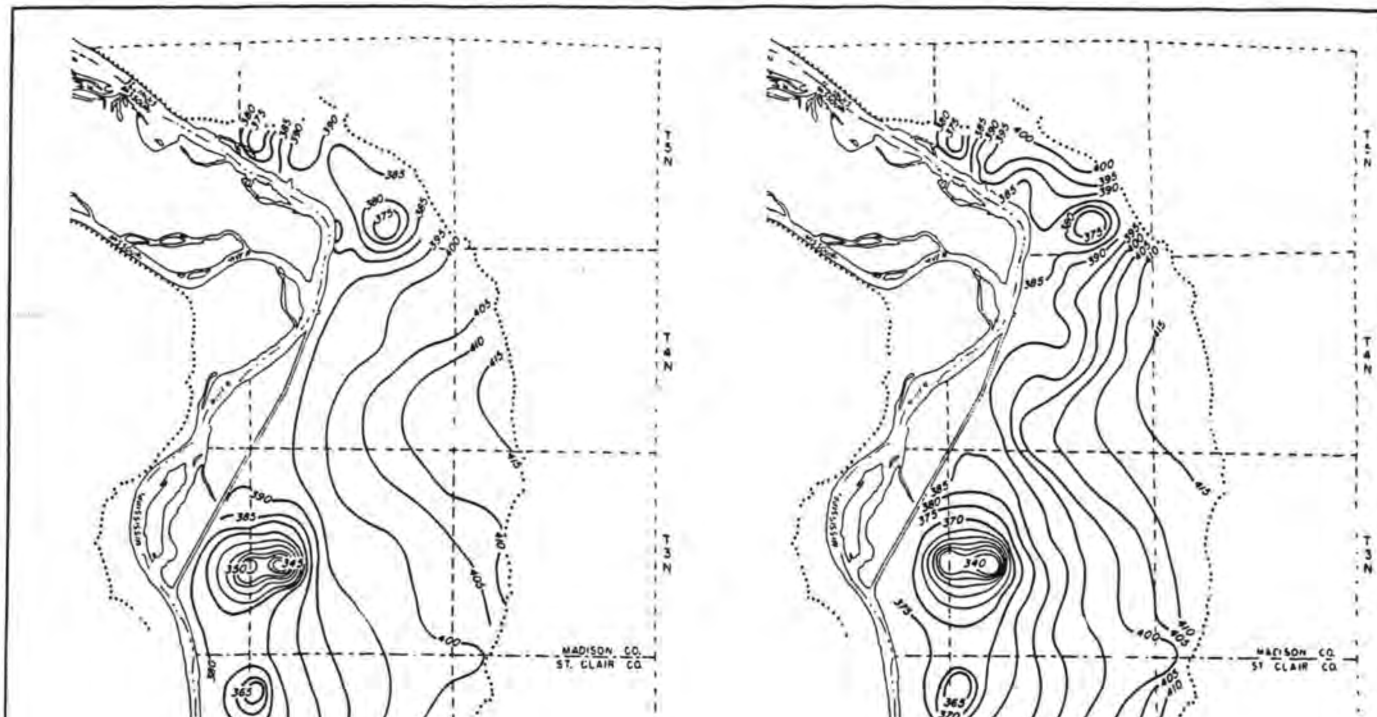


Figure 61. Elevation of piezometric surface, December 1956, actual (A), based on analog computer results (B)

indicate that stabilization of the piezometric surface during 1956 was mostly due to the effects of the Mississippi River. During much of the latter part of the drought there were long periods when little water was in the small streams and lakes in the interior portion of the East St. Louis area, and these hydrologic features had for practical purposes negligible influence on water levels.

Computations made with equation 4, taking into consideration the Mississippi River (recharge boundary) and accumulated periods of little or no recharge directly from precipitation, indicate that the piezometric surface for 1956 can be duplicated by using a time period of 5 years in estimating water-level declines.

Production wells were grouped into centers of pumping, and the average discharges during the period 1952-1956 for each pumping center were determined. The analog model was coupled to the excitation-response apparatus and the pulse generator was connected to junctions at locations of pumping centers. The output of the pulse generator was adjusted in accordance with discharge data and a maximum time period of 5 years. The oscilloscope was connected to terminals representing observation wells and water-level declines were computed. Thus, water-level declines everywhere in the aquifer were described. The total water-level decline, based on the superposition theorem, at each terminal was obtained by summation of individual effects of each pumping center. Only the effects of pumping centers were taken into account and the average stage of the Mississippi River was assumed to be the same in 1956 as it was in 1900. However, records show that the average stage of the Mississippi River was about 11 feet lower in 1956 than in 1900. The effect of the change in the average stage of the river on water levels was estimated by coupling the pulse generator to junctions in the analog model along the river and measuring water-level changes due to the given change of the stage of the river with the oscilloscope connected to junctions in the interior portions of the analog model.

The above water-level declines due to the decline in river stage were superposed upon water-level changes due to pumpage, and a water-level change map covering the period 1900 to December 1956 was prepared. A piezometric surface map (figure 61B) was constructed by superposing the water-level change map on the piezometric surface map for 1900.

Features of the piezometric surface map prepared with data from the analog computer and the piezometric surface map prepared from actual water-level data are generally the same, as shown in figure 61. A comparison of water-level elevations for selected pumping centers, based on the analog computer and actual piezometric surface maps, are given in table 31. The average slope of the piezometric surface in areas remote from pumping centers from both maps was 5 feet per mile. A comparison of gradients from analog computer and actual piezometric surface maps in the vicinity of pumping centers is given in table 32.

Table 31. Comparison of Analog Computer and Actual Piezometric Surface Maps for December 1956

Pumping center	Water-level elevation (ft above msl) Analog computer	Actual
Alton area	375	375
Wood River area	375	375
Granite City area	345	350
National City area	365	365
Monsanto area	360	355
Caseyville area	400	400

Table 32. Comparison of Analog Computer and Actual Hydraulic Gradients of Piezometric Surface Maps for December 1956

Pumping center	Average gradient (ft/mi) Analog computer	Actual
Alton area	15	15
Wood River area	15	15
Granite City area	20	30
National City area	10	10
Monsanto area	20	25

Differences in analog computer and actual piezometric surface maps are not significant when considered in relation to the accuracy and adequacy of geohydrologic data. The close agreement between analog computer and actual piezometric maps indicates that the analog computer may be used to predict with reasonable accuracy the effects of future ground-water development and the practical sustained yield of existing pumping centers.

PRACTICAL SUSTAINED YIELDS OF EXISTING PUMPING CENTERS

In 1962 water levels were not at critical stages in any pumping center and there were areas of the aquifer unaffected by pumping. Thus, the practical sustained yield of existing pumping centers exceeds total withdrawals in 1962. The practical sustained yield is here defined as the rate at which ground water can be continuously withdrawn from wells in existing pumping centers without lowering water levels to critical stages or exceeding recharge. Ground water withdrawn from wells less than 1 mile from the river was not considered.

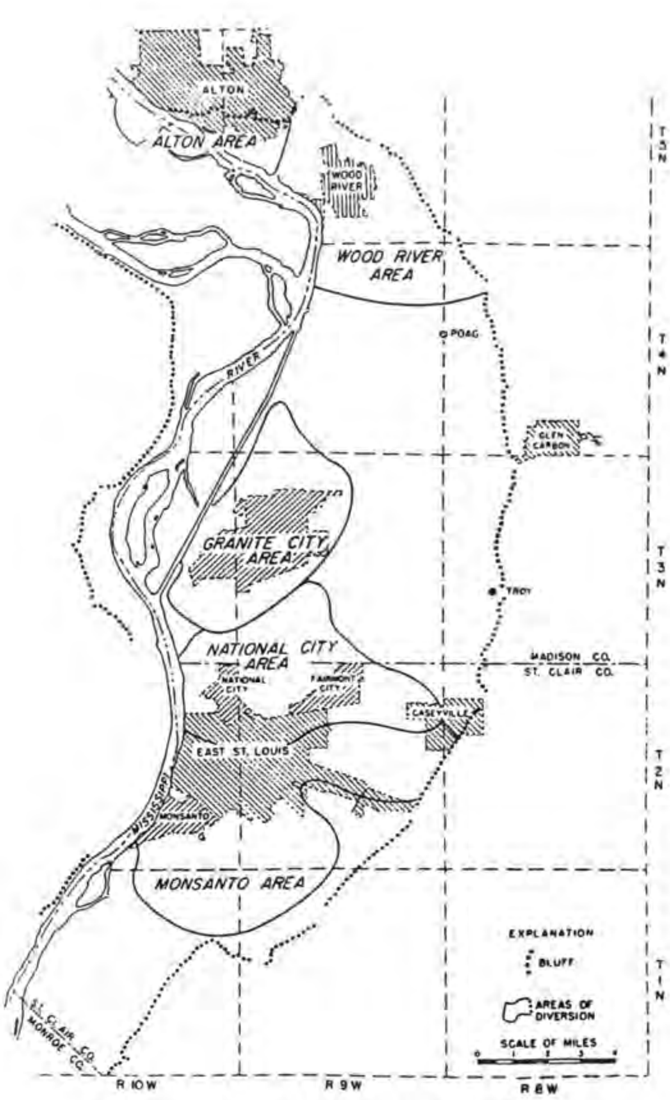


Figure 62. Areas of diversion in November 1961

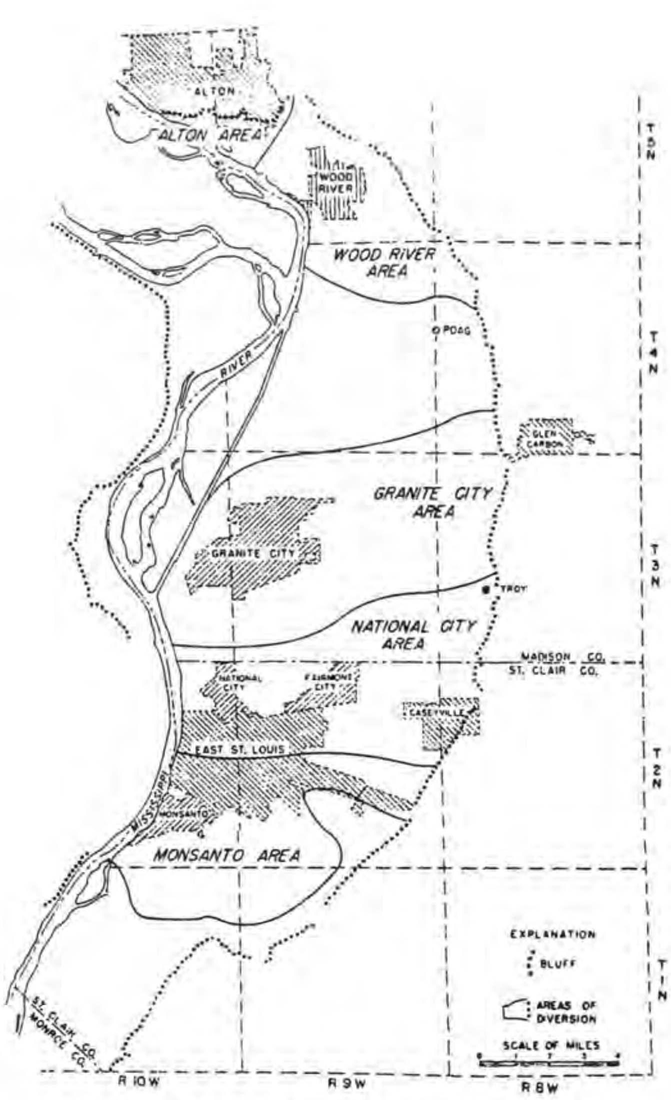


Figure 63. Areas of diversion in December 1956

Areas of diversion of pumping centers in November 1961 are shown in figure 62. The boundaries of areas of diversion delimit areas within which the general movement of ground water is toward production wells. The area (59 sq mi) north and east of Granite City and south of Wood River and a larger area south of Prairie Du Pont Creek through Dupo and south along the Mississippi River were outside areas of diversion. As shown in figure 63, the area north of Granite City outside areas of diversion was much smaller, covering about 30 sq mi, in December 1956. Pumpage in the Granite City area was 30.1 mgd in 1956 and 8.8 mgd in 1961.

Most of the coefficient of transmissibility of the valley fill deposits can be attributed to the coarse alluvial and valley-train sand and gravel encountered in the lower part of the valley fill. The thickness of the medium sand and coarser alluvial and valley-train deposits was determined from logs of wells and is shown in figure 64. The thickness of the coarse alluvial and valley-train sand and gravel exceeds 60 feet in an area south of Alton along the Mississippi River, in an area near Wood River, in places along the Chain of Rocks Canal, in a strip 1/2 mile wide and about 3 miles long through National City, in the Monsanto and Dupo areas, and in a strip about 1 mile wide and 4 miles long near Fairmont City. Thicknesses average 40 feet over a large part of the East St. Louis area. The coarser deposits diminish in thickness near the bluff, west of the Chain of Rocks Canal, and in places along the Mississippi River.

The available drawdown to the top of the medium sand and coarser deposits was estimated by comparing elevations of the top of the medium sand and coarser deposits with elevations of the piezometric surface map for June 1962 (figure 54). As shown in figure 64, available drawdown is greatest in undeveloped areas, exceeding 80 feet in the vicinity of Long Lake and in an area south of Horseshoe Lake. In a large part of the area available drawdown exceeds 60 feet. Average available drawdown within pumping centers was estimated to be 40 feet in the Alton area, 20 feet in the Wood River area, 35 feet

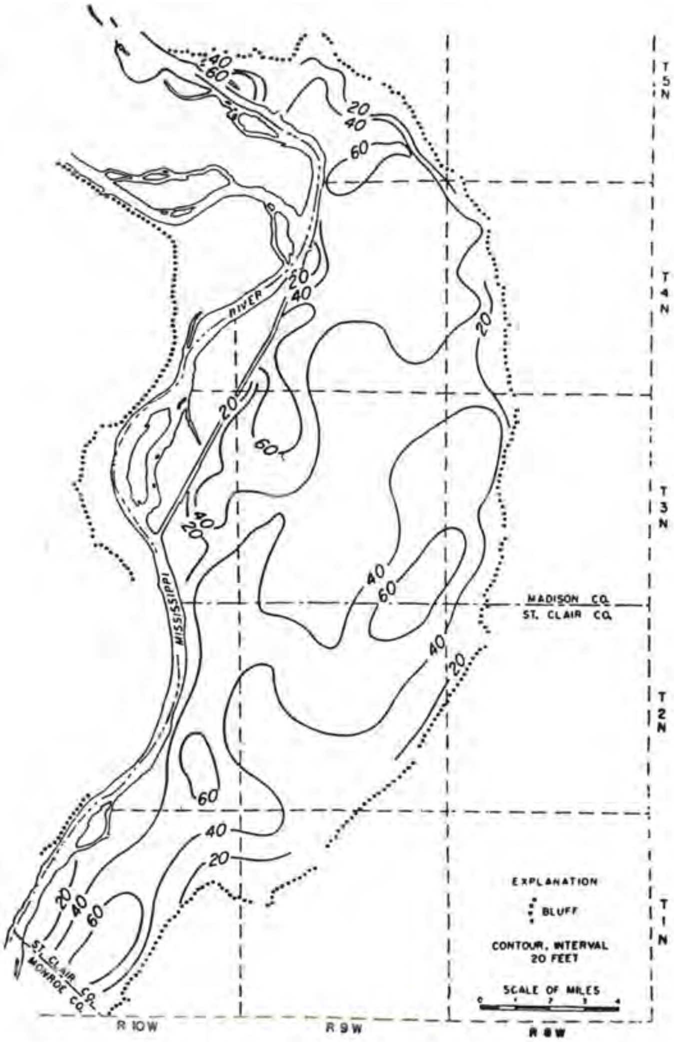


Figure 64. Thickness of medium sand and coarser deposits in lower part of valley fill

in the Granite City area, 30 feet in the National City area, and 30 feet in the Monsanto area.

When pumping water levels in individual production wells are below tops of screens, partial clogging of screen openings and the pores of the deposits in the immediate vicinity of the wells is greatly accelerated. To insure long service lives of wells, pumping water levels should be kept above tops of screens. Also, when water levels decline to stages below the top of the coarse alluvial and valley-train sand and gravel and more than one-half of the aquifer is dewatered, drawdowns due to the effects of dewatering become excessive and the yields of wells greatly decrease. Thus, critical water levels occur when pumping water levels are below tops of screens, or more than one-half of the aquifer is dewatered, or both.

Critical nonpumping water levels for existing pumping centers (table 33) were estimated on the basis of well-construction and performance data and figures 6, 64, and 65 taking into consideration the effects of dewatering. After critical water levels have been reached, individual wells in pumping centers will have yields exceeding 450 gpm.

Table 33. Critical Nonpumping Water-Level Elevations for Existing Pumping Centers

Pumping center	Average critical nonpumping water-level elevation (*ft above msl*)
Alton area	375
Wood River area	369
Granite City area	374
National City area	374
Monsanto area	369

The electric analog computer with a pumping period of 5 years was used to determine pumping center discharge rates that would cause water levels in all major pumping centers to decline to the critical stages in table 33. Several values of discharge were assumed and water-level declines throughout the East St. Louis area were determined. Water-level declines were superposed on the 1900 piezometric surface map together with changes in

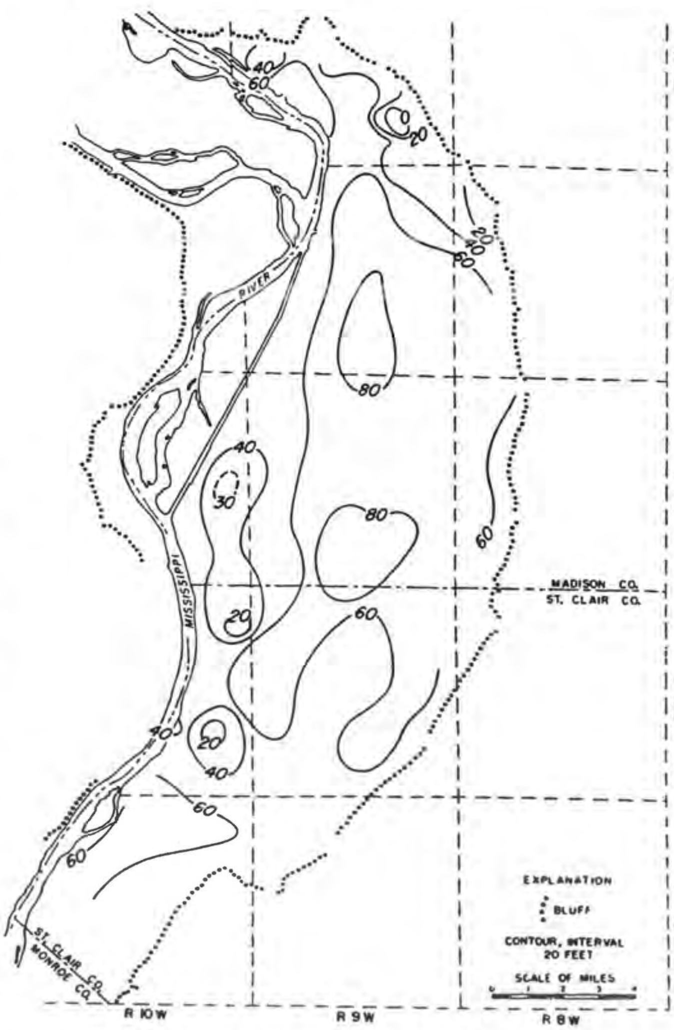


Figure 65. Estimated available drawdown to top of medium sand and coarser deposits in June 1962

water levels due to the changes in the stage of the Mississippi River, and piezometric surface maps under assumed pumping conditions were prepared. The pumping center discharge rates that resulted in a piezometric surface map with the critical water-level elevations in table 33 were assigned to the practical sustained yields of the pumping centers. The practical sustained yields of the existing pumping centers are given in table 34.

Table 34. Practical Sustained Yields of Existing Major Pumping Centers

Pumping center	1962 pumping rate (mgd)	Additional possible withdrawal (mgd)	Practical sustained yield (mgd)	Year after which practical sustained yield may be exceeded
Alton area	6.3	9.7	16	2000
Wood River area	14.1	5.9	20	1990
Granite City area	9.5	5.5	15	1980
National City area	11.6	6.4	18	2000
Monsanto area	22.6	0.4	23	1965
Total	64.1	27.9	92	

Estimates were made of the probable dates when practical sustained yields of existing pumping centers may be exceeded. Pumpage totals from 1890 through 1962 in the Alton, Wood River, Granite City, National City, and Monsanto areas are shown in figures 35 and 36. The past average rate of pumpage increase in each pumping center was estimated and extended to intersect the practical sustained yield of each pumping center. The assumption was made that the distribution of pumpage will remain the same as it was in 1962. It is estimated that the practical sustained yield of the Alton area pumping center (16 mgd) will be reached after the year 2000; the practical sustained yield of the Wood River area pumping center (20 mgd) will be reached about 1990; and the practical sustained yield of the Granite City area pumping center (15 mgd) will be reached about 1980.

It is estimated that the practical sustained yield of the National City area pumping center (18 mgd) will be reached about the year 2000. The rate of pumpage growth in the National City area may increase markedly, however, because of the effects of a series of drainage wells being installed to permanently dewater a cut along an interstate highway near National City. Pumpage from the drainage wells was not known at the time this report was prepared.

Pumpage in the Monsanto area during 1962 (22.6 mgd) is near the estimated practical sustained yield of 23 mgd.

No great accuracy is inferred for the estimated dates when practical sustained yields may be exceeded in table 34; they are given only to aid future water planning. A reasonable extrapolation of the pumpage graphs in figures 35 and 36 suggests that total ground-water withdrawals from wells in existing major pumping centers will exceed the practical sustained yields by about 2000.

POTENTIAL YIELD OF AQUIFER WITH A SELECTED SCHEME OF DEVELOPMENT

The electric analog computer was used to describe the effects of a selected scheme of development and to determine the potential yield of the aquifer under assumed pumping conditions. The potential yield of the aquifer is here defined as the maximum amount of water that can be continuously withdrawn from a selected system of well fields without creating critical water levels or exceeding recharge.

The distribution of pumpage with the selected scheme of development is shown in figure 66. A comparison of figures 66 and 34 shows that, with the exceptions of three new pumping centers near the river and one new pumping center in the Dupo area, the selected scheme of development is the same as the actual scheme of development in 1962.

Critical nonpumping water levels for existing and assumed pumping centers (see table 33) were estimated from figures 6, 64, and 65 taking into consideration the effects of dewatering. The electric analog computer was used to determine pumping center discharge rates that would cause water levels in all major pumping centers to decline to the critical stages in table 33. Several values of discharge in major pumping centers and anticipated discharge rates for minor pumping centers based on extrapolations of pumpage graphs for minor pumpage centers to the year 2015 were assumed and water-level declines throughout the East St. Louis area were determined. Model aquifers and mathematical models (Walton, 1962) based on available geohydrologic data and information on induced infiltration rates were used to determine the local effects of withdrawals in pumping centers near the river. Water-level declines were superposed on the piezometric surface map for 1900 together with changes in water levels due to the changes in the stage of the Mississippi River, and piezometric surface maps under assumed pumping conditions were prepared. The total pumping center discharge rate that resulted in a piezometric surface map with the critical water-level elevations in table 33 was assigned to the potential yield of the aquifer with the selected scheme of development. The potential yield, subdivided by pumping center, is given in table 35; water-level declines and approximate elevations of the piezometric surface with the selected scheme of development are shown in figures 67 and 68, respectively.

The pumpage graph in figure 32 was extrapolated into the future. Assuming that pumpage will continue to grow in the future as it has in the past, total pumpage in the East St. Louis area will exceed the potential yield with the selected scheme of development (188 mgd) after about 52 years or by 2015. A careful study of figures 25 and 66 and data on infiltration rates of the Mississippi River indicates that there are sites near the river where additional pumping centers could be developed. Thus, the potential yield of the aquifer with other possible schemes of development exceeds 188 mgd.

Recharge by Source

Flow lines were drawn at right angles to piezometric surface contours in figure 68 and areas of diversion (see figure 69) of pumping centers were delineated. Recharge directly from precipitation to each pumping center was computed as the product of areas of diversion and the average recharge rate (370,000 gpd/sq mi). Recharge from subsurface flow through the bluffs to each pumping center was computed as the product of the lengths of the bluff within areas of diversion and the average rate of subsurface flow (329,000 gpd/mi). Recharge from induced infiltration of surface water in the Mississippi River to each pumping center was determined by subtracting the sums of recharge directly from precipitation and subsurface flow from discharge rates in table 33. Recharge subdivided by source is given in table 36.

It is estimated that 36.5 percent of the total potential yield of the aquifer with the selected scheme of development will be derived from recharge directly from precipitation; about 57.3 percent will be derived from recharge by induced infiltration of surface water; and about 6.2 percent will be derived from recharge by subsurface flow through the bluffs.

Recharge amounts in 1956 and 1961, subdivided by source, are also given in table 36. The percentage of recharge from induced infiltration of surface water increases as the total withdrawal rate increases. As shown

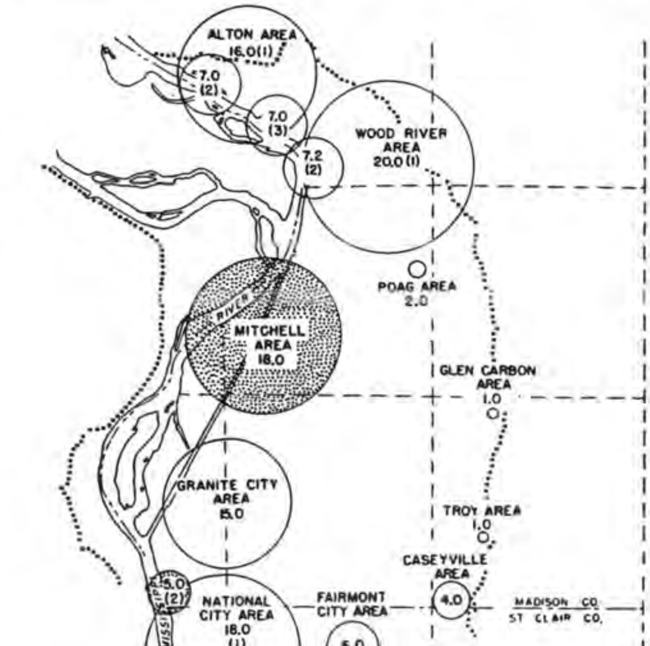


Figure 66. Distribution of pumpage with selected scheme of development

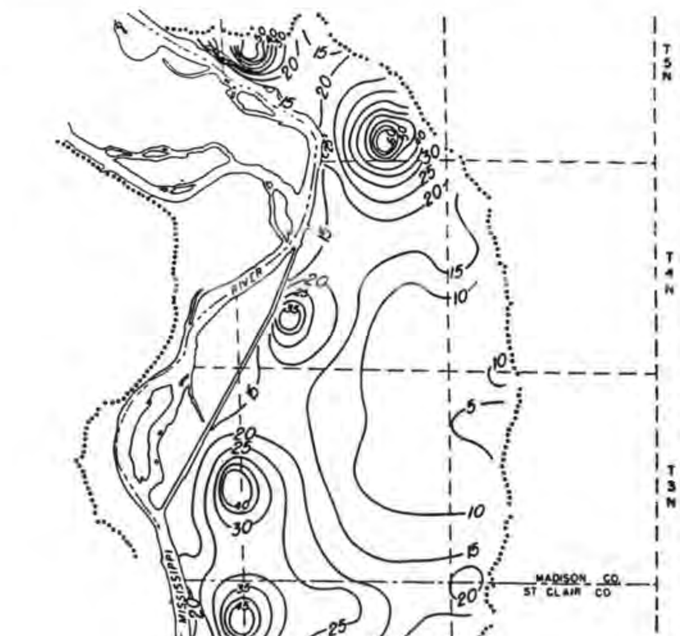


Figure 67. Water-level declines with a selected scheme of development

Table 35. Potential Yield of Aquifer with a Selected Scheme of Development

Pumping center	Pumpage with selected scheme of development (mgd)
Alton area	
1	16.0
2	7.0
Wood River area	
1	20.0
2	7.2
3	7.0
Mitchell area	18.0
Granite City area	15.0
National City area	
1	18.0
2	5.0
3	7.5
Monsanto area	
1	23.0
2	11.0
Dupo area	19.0
Poag	2.0
Glen Carbon	1.0
Troy	1.0
Caseyville	4.0
Fairmont City	6.0
Total	187.7

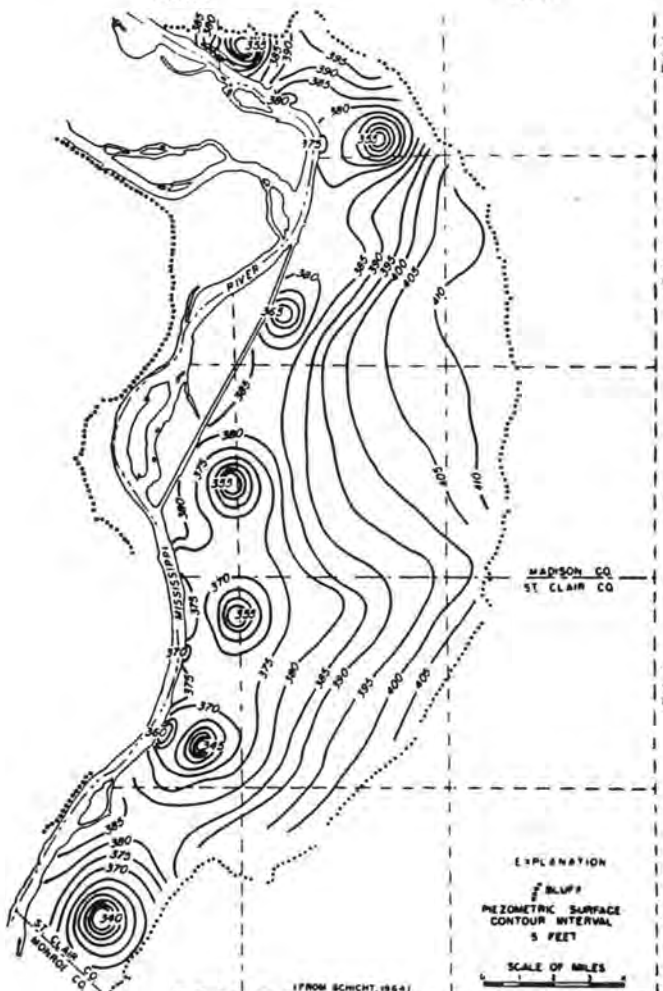


Figure 68. Approximate elevation of piezometric surface with a selected scheme of development

in figure 69 areas of diversion with the selected scheme of development cover most of the East St. Louis area. Recharge directly from precipitation and subsurface flow through bluffs is therefore nearly at a maximum. Additional pumpage will have to be balanced with recharge mostly from induced infiltration of surface water. This can best be accomplished by developing additional well fields near the Mississippi River.

Average head losses beneath the Mississippi River bed and river-bed areas of induced infiltration, associated with pumpage in 1962 and with the selected scheme of development, were estimated based on infiltration rates and aquifer-test data. Average head losses are much less than the estimated depths of the Mississippi River given in figure 57, and river-bed areas of induced infiltration are small in comparison to the river-bed area in the East St. Louis area, indicating that recharge from the induced infiltration of surface water with the selected scheme of development is much less than the maximum possible induced infiltration.

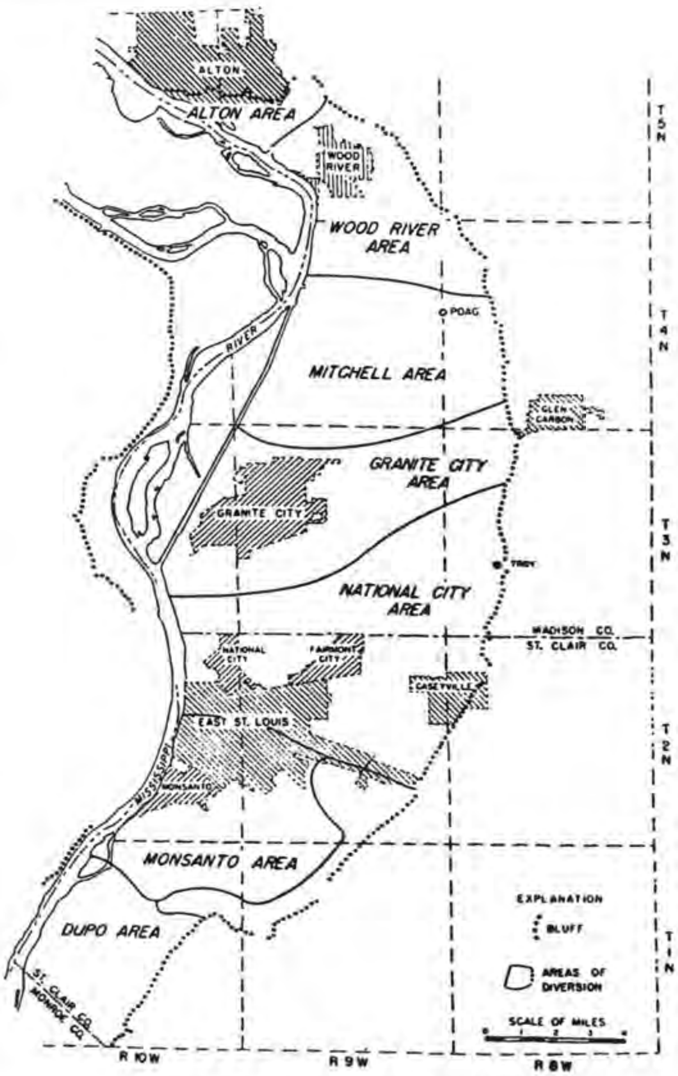


Figure 69. Areas of diversion with selected scheme of development

Table 36. Recharge with Selected Scheme of Development and in 1956 and 1961, Subdivided by Source

Pumping center	Selected scheme of development				1956				1961			
	Precipitation (mgd)	Subsurface flow (mgd)	Induced infiltration (mgd)	Total pumpage (mgd)	Precipitation (mgd)	Subsurface flow (mgd)	Induced infiltration (mgd)	Total pumpage (mgd)	Precipitation (mgd)	Subsurface flow (mgd)	Induced infiltration (mgd)	Total pumpage (mgd)
Alton area	1.8	2.0	19.2	23.0	1.4	1.4	7.0	9.8	1.0	1.1	10.2	12.3
Wood River area	7.3	2.6	24.3	34.2	7.1	3.0	11.0	21.1	7.2	2.6	14.5	24.3
Mitchell area	9.3	1.3	7.4	18.0								
Granite City area	11.2	0.9	2.9	15.0	16.8	1.8	*	30.1	7.7		1.1	8.8
National City area	9.9	2.0	18.6	30.5	9.0	1.1	3.7	13.8	6.9		3.9	10.8
Monsanto area	9.5		24.5	34.0	10.7	0.7	18.7	30.1	12.6	0.8	18.5	31.9
Dupo area				19.0								
Poag	2.0			2.0	0.9			0.9	1.2			1.2
Glen Carbon	0.8	0.2		1.0	0.2			0.2	0.3			0.3
Troy	0.8	0.2		1.0	0.3			0.3	0.4			0.4
Caseyville	2.9	1.1		4.0	1.3	1.0		2.3	1.4	1.0		2.4
Fairmont City	6.0			6.0	2.4			2.4	4.4			4.4
Total	61.5	10.3	96.9	187.7	50.1	9.0		111.0	43.1	5.5	48.2	96.8

**Not computed; water being taken out of storage*

WATER QUALITY

The chemical character of the ground-water in the East St. Louis area is known from the analyses of water from 183 wells. The results of the analyses are given in table 37. The constituents listed in the table are given in ionic form in parts per million. The analyses of water from wells were made by the Chemistry Section of the State Water Survey. Chemical analyses of water from wells at several sites in the area are made monthly by the chemistry section. The locations of selected sites are given in figure 70. The sampling periods are listed in table 38, which provides a summary of the results of periodical chemical analyses of water from selected wells.

Ground water in the East St. Louis area varies in quality at different geographical locations. The quality of water also varies with the depth of wells, and may often be influenced by the rate of pumping and the idle period and time of pumping prior to collection of the sample. Bruin and Smith (1953) noted that relatively shallow wells of a depth less than 50 feet are in general quite highly mineralized and frequently have a high chloride content. Water samples from wells in heavily pumped areas often have high sulfate and iron contents and a high hardness.

Induced infiltration of water from the Mississippi River affects the chemical quality and temperature of water in wells at many sites. All other factors being equal, the closer the well is to the river the greater will be the effect of induced infiltration on the quality and temperature of water in the well. In most of the analyses in tables 37 and 38 the effect of induced infiltration of river water is not evident. Data in figure 71 illustrate the effect of induced infiltration of water from the river

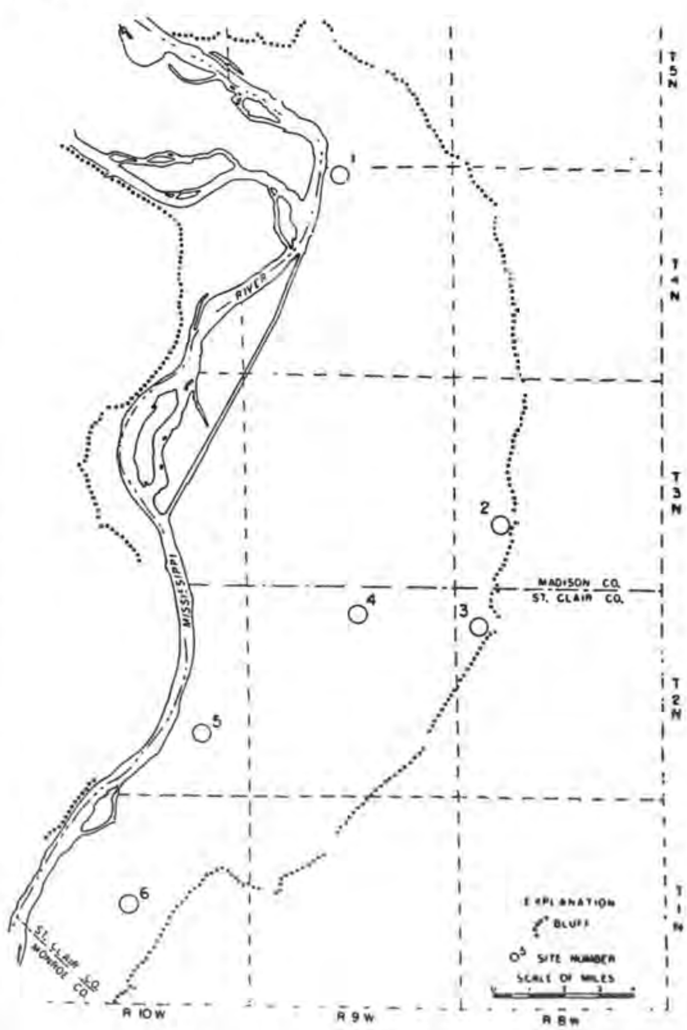


Figure 70. Sites where ground-water samples are periodically collected

Table 37. Chemical Analyses of Water from Wells

(Chemical constituents in parts per million)

Well number	Depth (ft)	Date collected	Silica (SiO_2)	Iron (Fe)	Manganese (Mn)	Calcium (Ca)	Magnesium (Mg)	Sodium + potassium (Na + K)	Boron (B)	Alkalinity (as $CaCO_3$)	Sulfate (SO_4)	Chloride (Cl)	Fluoride (F)	Nitrate (NO_3)	Hardness (as $CaCO_3$)	Total dissolved minerals	pH	Temperature (°F)
MAD—																		
5N9W-16.		12/16/48	33.6	0.2	0.3	108.7	30.5	23.9		224	171.1	27.0		11.5	398	538		
5N9W-18.4b	89	11/30/58	23.0	8.2	0.0	156.5	43.0			434	137.5	31.0		0.9	434	593		
5N9W-18.5c	93	3/31/54	12.0	2.0		123.0	61.4			314	191.3	27.0		3.7	314	560		
5N9W-22.		4/ 1/42	22.0	0.5	0.4	76.6	26.3	16.8		254	65.2	8.0		3.4	299	384	7.2	55
5N9W-		12/ 3/48	28.7	0.2	0.3	101.3	37.6	3.2		300	99.6	5.0	0.2	6.1	408	473		56
5N9W-		6/ 7/58	31.3	0.4		104	36.9	16	0.1	308	117.6	11	0.1	1.2	413	417		56
5N9W-	92	8/12/59		0.6	0.5					324		17	0.1	1.5	450	524		55
5N9W-26.7c	112	4/20/50		0.4	0.3	66.0	22.2	0.7		192	59.2	3.0	0.3	0.1	256	329		59
5N9W-26.8e	114	4/20/50	30.9	0.7	0.3	90.0	26.3	0.7		244	83.3	3.0	0.3	0.3	333	400		57
5N9W-26.8g	110	3/22/57	21.3	0.3	0.6	110.1	30.8	14.0	0.1	276	140.5	8.0	0.1	0.8	402	516		58
5N9W-26.8g	116	12/10/48	27.4	0.4	0.2	64.2	20.6	2.8		176	65.2	5.0	0.3	0.6	246	304		57
5N9W-27.1b1	122	12/ 4/48	31.3	4.8	0.5	87.0	25.0	7.6		200	105.9	19.0	0.6	0.2	321	410		57
5N9W-28.	86	11/ 1/60		5.8						284	50.4	15			335	406		55
5N9W-33.4d	100	12/16/58	11.1	1.3	0.3	52.5	18.2	14	0.0	160	45.3	20	0.1	0.9	206	270		
5N9W-33.5f	80	6/15/60		0.2	0.3					164	54.1	18		1.5	221	261		58
5N9W-35.4g	113	3/29/46		18.4						216	264.7	90.0			546	711		58
5N9W-35.4h	135	3/29/46		11.9						292	209.4	6.0			485	527		58
5N9W-35.3h	126	3/29/46		12.5						284	176.9	11.0			450	502		58
4N8W- 6.4a	71	8/30/56		6.2					0.3	556		11			448	558		58
4N8W-19.2g	1300	2/14/60		10						324	2137.8	4050			920	10373		
4N8W-19.7h	37	10/22/59	40.8	4.2	Tr	40.9	13.5	7	0	108	36.6	6	0.1	22.7	158	258		59
4N8W-19.8e	95	12/14/60		1.0						256		2	0.2	0.4	316	389		56
4N8W-29.4a	41	9/ 5/57		1.2	1.3					280		6	0.2	1.3	356	393		55
4N8W-29.4h	115	9/16/54		0.6					0	344		6			468	506		56
4N9W- 1.4d	100	6/31/61		1.6					0	236		3			289	334		
4N9W- 9.2b	85	11/10/52		2.0						308	113.1	8			420	472		
4N9W-12.5g	40	8/ 3/60		0.1						168		2		10	220	243		58
4N9W-13.	111	2/40	24.0	2.0		52.2	8.0	19.1		146	44.0	3.0		11.2		163	7.1	
4N9W-	112	11/17/48	27.1	2.8	0.3	59.9	15.0			144	51.8	5.0	0.5	8.1	211	264		57
N9W-	114	11/ 6/61		1.6	0.3	60.0	15.6			152		4.0	0.2	9.8	214	271	7.7	58
N9W-	117	10/26/61		2.6	0.3	65.9	17.8			180	58.4	5	0.2	9.0	238	300	7.6	59
N9W-16.3b	35	10/21/43		0.1						334		49			914	1169		
4N9W-16.3a1	27	10/21/43		Tr						334		56			895	1179		
4N9W-16.3c2	85	9/ 7/53		5.9						224	15.1	4			224	251		
4N9W-16.5b1	22	11/28/49		4.5						380	52.5	13		0.5	438	473		
4N9W-16.5c	72	1/27/53		5.9						280	43.4	3			316	339		
4N9W-19.3b1	26	1/ /44		0.2						446		26			628	1000		
4N9W-19.3b2	60	6/ 8/61		0.1						472	186.2	27		51.8	628	865		55
4N9W-20.3h	69	8/21/52		4.6						272	74.5	5			344	358		56
4N9W-20.4e		6/12/52		10.7						276	41.8	3			315	332		56
4N9W-20.4f		6/14/52		11.5						316	53.1	3			348	377		56
4N9W-20.4g		6/11/52		9.2						320	63.8	4			391	396		56
4N9W-21.5h	106	10/15/52	33.0	3.4	0.3	62.4	17.1	10.4		244	1.6	2	0.2	0.4	226	268		
4N9W-27.1f	110	10/14/43		3.5						242		8			291	345		60
4N9W-29.7c	106	4/27/54		9.4					0.1	332		3			370	380		56
4N9W-29.7e	63	9/18/52		12.1						328	79.0	6			418	418		58
4N9W-29.7g	67	9/16/52		11.0						356	79.4	3			420	435		57
4N9W-29.8e	63	9/19/52		13.0						340	66.0	5			412	412		56
4N9W-30.1a	69	9/25/52		13.0						352	72.7	6			428	430		57
4N9W-30.1b	69	9/25/52		10.3						360	87.4	6			448	458		57
4N9W-30.1c	66	9/24/52		10.1						340	80.8	9			428	433		56
4N9W-30.2a	69	8/28/52		9.5						376	56.2	4			424	424		57
4N9W-31.2g	69	8/26/52		10.8						364	76.7	5			433	440		57
4N9W-31.2h	69	8/27/52		13.0						392	90.7	5			466	492		56
4N9W-31.3f	71	8/23/52		10.9						404	71.6	5			458	486		57
4N9W-31.3g	68	8/25/52		10.8						396	98.1	8			487	515		56
4N9W-31.5b	63	7/28/52		7.6						336	43.2	5			378	392		57
4N9W-31.6a	60	8/ 5/52		10.3						348	64.4	6			416	427		57
4N9W-31.6b	58	7/25/52		9.1						352	50.6	5			395	402		57
4N9W-33.1g	110	11/21/53		5.1						296	2.5	6			268	304		
3N8W- 5.2f1	66	11/ 3/51	21.6	0.9	0.4	112.9	47.0	12.4		244	223.6	13	0.4	9.9	476	590		57
3N8W- 5.2f2	63	4/28/58	20.3	0.4	0.2	119.2	52.7	23	0.2	276	251.8	17	0.3	1.9	515	666		57
3N8W- 8.4h1	41	9/ 9/58		0.1						324		24			540	594		56
3N8W-20.5c1	100	6/30/59		0.2						320	115.8	6			424	478		57
3N8W-20.5c2	45	9/21/55		Tr					0	292		4			388	392		57
3N8W-20.8c1	100	10/ 7/43		0.1						316		7			336	365		62
?N8W-20.8c2	95	6/30/59		0.8						276	71.0	6			339	424		58
N8W-29.3h	115	7/26/57	25.0	0.1	0.1	102.5	44.4	9	0.3	292	148.9	7	0.1	1.3	438	517		56
N8W-30.7b1	40	10/13/54		4.2						324	49	3			340	369		
N8W-30.7b2	104	10/13/54		1.8					0.6	296		4	0.3		332	364		
3N8W-31.2a1	99	4/ 3/52	27.0	1.4	0.9	107.6	43.8	8.1		280	170.5	6	0.4	0.6	449	552		56
3N8W-31.2a2	102	8/17/55	35.0	0.8	0.4	87.4	35.9	4	0.1	316	47.3	7	0.3	0.7	366	412		57
3N8W-31.2a3	103	8/12/58		1.6	0.5					280		18	0.1	1.0	450	465		56
3N9W- 3.	110	2/18/44		9.1						322		2.0			349	377		
3N9W- 5.8h	110	4/27/54		6.6					0	264		6			336	347		56

Table 37 (Continued)

Well number	Depth (ft)	Date collected	Silica (SiO_2)	Iron (Fe)	Manganese (Mn)	Calcium (Ca)	Magnesium (Mg)	Sodium + potassium (Na + K)	Boron (B)	Alkalinity (as $CaCO_3$)	Sulfate (SO_4)	Chloride (Cl)	Fluoride (F)	Nitrate (NO_3)	Hardness (as $CaCO_3$)	Total dissolved minerals	pH	Temperature (°F)
MAD—(Continued)																		
3N9W- 6.3c	110	5/13/54		7.2					Tr	276		4			352	364		57
3N9W- 6.4a1	32	9/25/54		0						312	212.4	12			525	587		56
3N9W- 6.4a2	56	1/29/57		7.8					0	300		4			424	468		58
3N9W- 6.8f	56	8/18/52		4.1						252	60.5	3			248	323		56
3N9W- 6.8g	55	8/14/52		3.0						244	58.2	4			294	317		58
3N9W- 6.8h	59	8/12/52		3.5						320	77.8	5			395	408		57
3N9W- 8.5g	80	9/23/51	41.8	8.1	0.2	114.2	28.8	2.8		240	143.3	15	0.5	0.5	404	486		57
3N9W-10.4g	27	1/ /44		2.7						316		13			444	518		
3N9W-10.4h	23	9/ 7/55		1.8						240		4			300	439		58
3N9W-14.2g	100	11/ 2/56		4.5					0.2	232		4			256	287		57
3N9W-17.2a	106	4/21/54		5.7					0.1	296		8			384	380		56
3N9W-18.1d	90			9.4						256		5	0.1	0.7	312	338		60
3N9W-18.1f	45	2/24/44		5.2						258		4			326	356		60
3N9W-19.8h	109			7.0						288	94.6	40			372	459		
3N9W-24.4g	104	9/ 7/54		11					0.3	320	38.3	5			336	356		57
3N9W-30.7e	101	6/ 9/58		8.7						308	138.6	29	0.3		420	520		57
3N9W-32.6g	30	9/22/60		1.5						336	162.1	22		0.7	510	613		54
3N9W-35.2d	100	11/ 1/55		1.6					0.1	344		10			400	422		57
3N9W-35.4a	28	3/27/34	14	0.0	0.35	197.8	43.3			326	146.0	111.0		88.6	320	673		
3N9W-35.4d	55	3/27/34	14	1.3	0.8	92.4	21.6	12.7		286	46.7	0		0.9		320		
3N9W-35.7d	65	6/ 3/53		8.2				6.7		180		50			124	295		57
3N9W-35.8a	100	11/ 6/45		7.6						304		5			335	368		58
3N10W- 1.1c	53	7/21/52		4.0						320	65.2	6			365	392		56
3N10W- 1.1d1	52	7/22/52		6.4						304	61.7	6			357	384		57
3N10W-1.1d2	72	11/21/53		10.1						376	138.2	25			528	580		56
3N10W- 1.2b	52	7/16/52		3.3						292	60.3	5			340	363		56
3N10W- 1.2c	53	7/18/52		2.2						328	76.5	4			386	405		58
3N10W- 1.3a	58	7/12/52		5.2						280	66.4	5			349	371		58
3N10W- 1.3b	58	7/15/52		3.3						312	51.6	4			361	375		57
3N10W-12.3d	65	3/ 2/61		7.0						404	193.4	12	0.3	0.8	590	699		57
3N10W-12.4g	57	7/ 2/52		4.8						196	74.7	8			265	297		56
3N10W-12.5f		6/26/52		5.5						204	82.5	8			276	318		58
3N10W-12.5e	57	6/28/52		7.2						200	103.7	8			312	335		57
3N10W-12.6d		6/20/52		8.4						232	93.2	7			320	358		57
3N10W-14.2d		9/ 2/52		9.6						268	76.5	9			336	364		57
3N10W-14.4a		9/ 6/52		8.4						204	78.0	10			286	319		60
3N10W-14.4b		9/23/52		8.7						216	68.7	11			300	316		55
3N10W-23.5e		7/15/52		9.8						276	102.0	9			365	400		56
3N10W-23.5h		9/ 9/52		8.4						224	72.4	8			286	320		53
3N10W-23.6e		9/13/52		10.8						324	129.2	13			441	500		60
3N10W-24.3h	84	2/23/44		8.8						334		38			712	825		56
3N10W-25.	119	2/24/44		16.6						362		37			720	891		60
3N10W-25.8g	95	6/26/52	45.0	22.6	3.0	162.0	39.3	35.9		344	257.9	24	0.3	0.1	567	757		
3N10W-36.5g	100	8/23/53		8.9						292		31			628	674		60
STC—																		
2N8W- 6.5h	106	4/29/55		0.6					Tr	296		7			388	408		56
2N8W- 6.6a	105	9/ 2/54		1.6					Tr	312		15			656	675		58
2N8W-7.2bl	91	4/12/35	12.0	0.6	0.6	156.8	77.7	71.5		461	316.1	52		1.3	708	1001		
2N8W- 7.3h	80	6/25/54		0.1					0.6	372	350.5	42			756	877		57
1N10W- 3.5e	31	11/ 2/43		0.2						386		71			960	1122		
1N10W- 9.1g	26	11/ 8/43		0.1						380		68			941	1123		
1N10W-12.7d	27	11/ 5/43		0.4						280		14			461	527		
1N10W-19.2h	34	11/16/43		0.1						412		141			1152	1566		
1N10W-21.4f	102	6/ 4/43		14.0						354		27			518	625		
1N10W-28.3h	51	6/21/43		1.4						120		16			322	386		
1N10W-33.7b	33	11/15/43		0.5						86		48			157	246		
2N9W- 1.3f	90	8/15/62		5.1	0.8					328		70	0.2	0.3	520	644		
2N9W- 1.4a	115	3/ /54		1.9					Tr	296		7			388	408		56
2N9W- 2.4f	36	11/ 6/45		3.6						280		10.0			338	382		
2N9W- 2.8e		9/27/45	32.6	7.5	0.5	94.4	29.6	1.6		324	29.6	5.0	0.3	0.2	358	406		
2N9W- 3.8a	115	4/ 8/43		6.2						382		5.0			408	450		
2N9W- 3.8a	117	4/ 8/43		6.6						290		12.0			617	742		
2N9W- 4.3a	112	4/ 1/43		7.6						312		52.0			938	1035		
2N9W- 4.3b	105	3/30/43		4.2						270		4.5			919	1176		
2N9W- 4.3c	122	3/30/43		6.2						314		13.0			401	477		
2N9W- 4.4a	100	4/ 1/43		12.4						270		42.5			1273	1882		
2N9W- 7.6e1	106	3/10/58		5.8	0.4	190	52			326		108			690	1014		58
2N9W- 7.6e2	100	3/10/58		2.2	0.4					336		110			700	1021		58
2N9W- 9.1h		2/ 2/48		14.8						324	32.1	5			353	369		
2N9W- 9.7a	98	5/14/54		7.3						352	97.9	15			452	492		58
2N9W-10.6h1	122	8/ /42		4.7						356		4.0			343	387		
2N9W-10.6h2	124	8/ /42		9.4						380		3.0			350	397		
2N9W-17.2d	108	4/ /54		6.8						308		7			360	387		
2N9W-17.7g	114	8/24/48		10.1						308	111.3				425	463		
2N9W-18.5b	116	3/25/43		7.1						336		39			612	725		60
2N9W-19.	104	3/19/43		11.0						364		36			608	778		54
2N9W-26.7f	81	9/ 7/51	36.8	2.1	0.3	84.3	31	11.5		228	118.1	11	0.4	0.8	342	437		57
2N9W-26.8f	81	8/31/51	36.0	2.6	0.2	69.4	26	11.5		188	97.1	13	0.4	0.6	283	358		57

Table 37 (Continued)

Well number	Depth (ft)	Date collected	Silica (SiO_2)	Iron (Fe)	Manganese (Mn)	Calcium (Ca)	Magnesium (Mg)	Sodium + potassium (Na + K)	Boron (B)	Alkalinity (as $CaCO_3$)	Sulfate (SO_4)	Chloride (Cl)	Fluoride (F)	Nitrate (NO_3)	Hardness (as $CaCO_3$)	Total dissolved minerals	pH	Temperature (°F)
STC—(Continued)																		
2N9W-28.6e	109	9/20/37	8.0	7.5		107.3	23	6.7		358		2.0		1.3	363	371		58
2N9W-29.6e	113	3/19/43		7.1						340		9.0			366	426		
2N9W-30.5h	100	3/17/43		25.0						420		33			590	850		
2N9W-30.6d	110	8/ 7/44		9.1						312		19			406	555		
2N10W- 1.3a4	110	5/16/61		11	0.8					304	149.3	16		1.9	444	569		57
2N10W- 1.3a5	110	5/16/61		14	1.0					296	146.5	13		0.7	436	551		57
2N10W-12.3c	106	9/ /54		12						396	537.7	170			884	1424		60
2N10W-12.3g2	108	3/30/43		5.6						418		640			1050	2258		
2N10W-12.3g3	108	do		3.8						404		225			803	1213		
2N10W-12.3g4	108	do		7.1						400		530			844	1810		
2N10W-12.6f	106	1/29/59		12						362	209.6	59			561	767		
2N10W-12.6h	100	1/29/59		15						436	209.6	92			625	913		
2N10W-13.6a	110	11/16/43		4.7						286		5			357	386		
2N10W-13.5d	108	3/17/43		12.8						370		53			682	840		
2N10W-13.7g1		9/ /44		12.2						352		48			686	882		
2N10W-13.7h2	38	9/ /44		0.9						368		33			616	803		
2N10W-24.1e		4/24/36	16.0	0.6	0.5	154.8	40.8	9.0		290	226.4	34		1.2	554	720		
2N10W-25.7b	100	4/ 1/43		1.7						396		32			322	469		54
2N10W-26.1e1	95	8/18/43		6.1						404		9			377	443		56
2N10W-26.1e2	105	12/12/47	37.0	12.8	0.3	130.2	40.6	15.6		360	137.0	18	0.3	0.1	493	603		
2N10W-26.2e	107	4/16/43		21.6						462		61			777	1108		60
2N10W-26.3d1	95	6/24/43		8.0						374		29			533	668		60
2N10W-26.3d2	110	6/24/43		11.3						328		23			518	676		60
2N10W-26.3g	105	12/12/47		22.9						440	486.7	41			750	1256		59
2N10W-26.3h2	105	6/10/43		1.1						386		39			770	864		
2N10W-26.3h3	105	6/10/43		12.4						402		39			770	890		
2N10W-26.4e	109	12/12/47	45.2	15.2	0.5	141.7	37	30.6		356	163.7	34	0.4	Tr	508	662		58
2N10W-26.7b	112			15.6						416	161.9	30			567	677		57
2N10W-26.7b	110	5/17/43		6.6						340		32			561	634		55
2N10W-33.2f	100	2/18/44		15.4						444		50			620	740		59
2N10W-34.	73	6/23/43		12.0						354		43			466	638		57

n water in a collector well owned by the Shell Oil Company located west of Wood River immediately adjacent to the river. The average monthly range in temperature of water in the collector well varies from about 50F during the late winter and early spring months to about 70F during the late summer and early fall months. Temperatures of the river water vary from a low of about 34F during January and February to a high of about 84F during July and August. The highs and lows of the temperature of the water from the collector well lag behind corresponding highs and lows of the temperature of the river water by 1 to 2 months, as shown in figure 71. During the period November 1953 to March 1958 the average monthly total hardness of water from the collector well varied from a low of 180 to a high of 253 ppm. During the same period the average total hardness of the river varied from a low of 150 to a high of 228 ppm. In general the water from the collector well is less hard than water in wells away from the river.

The hardness of waters in the East St. Louis area, as indicated in table 37 ranges from 124 to 1273 ppm and averages 459 ppm. In general, water in excess of 500 ppm hardness is found in wells less than 50 feet in depth. The iron content ranges from 0 to 25.0 ppm and verages 6.2 ppm. The chloride content ranges from 0 o 640 ppm and averages 27 ppm. Fluoride content ranges from 0.1 to 0.6 ppm.

The temperature of water from 121 wells in the sand and gravel aquifer ranges from 53 to 62F and averages 57.3F. A seasonal variation in temperatures of water in wells is not readily apparent.

Chemical analyses and temperatures of water from the Mississippi River at Alton and Thebes, Illinois, are given in tables 39 and 40 respectively.

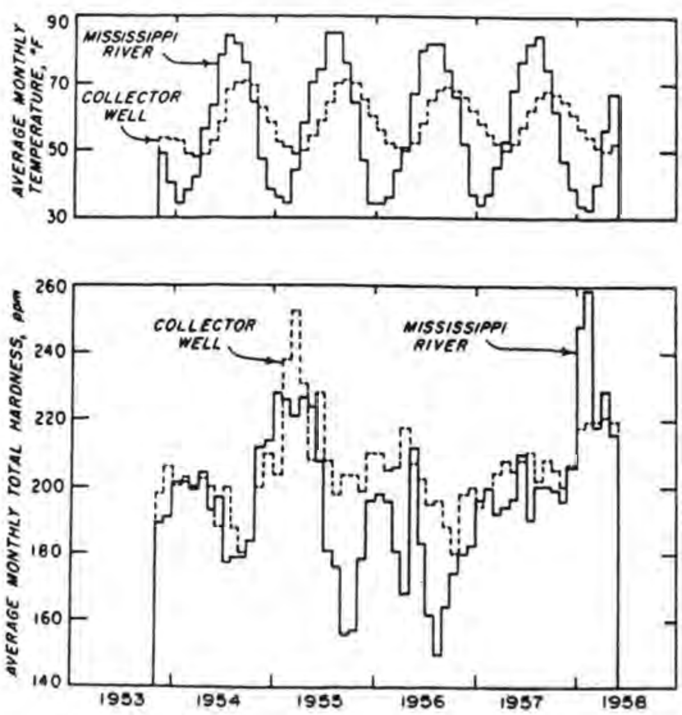


Figure 71. Chemical and temperature data for collector well and Mississippi River, 1953-1958

Table 38. Summary of Results of Periodical Chemical Analyses for Selected Wells

(Chemical constituents in parts per million)

Owner	Site number	Iron (Fe)	Chloride (Cl)	Sulfate (SO_4)	Alkalinity (as $CaCO_3$)	Hardness (as $CaCO_3$)	Total dissolved minerals	Temperature (°F)	Well number and period of record
Western Fibre Co.	3	0.9- 3.6	165- 260	388.8- 496.4	572- 872	956- 1140	1400- 1894	59- 61	Well 3 6/25/56 to 1962
	3	1.6- 6.8	150- 220	371.3- 485.5	408- 776	820- 1100	1202- 1761	58- 64	Well 1 10/31/49-5/28/56
Hartford (V)	1	2.3- 12.2	5- 30	43.4- 129.8	204- 384	247- 482	336- 580	55.5 61	Well 2 7/5/47 to 1962
Virginia Carolina Chemical Co.	4	2.3- 50	2- 22	1.4- 1130.1	272- 456	331- 1560	353- 2075	55- 60	7/10/44 to 1962
Missouri and Pacific R. R.	6	5.4- 15.0	9- 28	52.2- 235.5	364- 428	420- 532	473- 731	57- 62	South well 10/28/59-9/3/58
	6	0.4- 20.8	2- 93	55.6- 374.2	76- 504	129- 733	240- 963	57- 62.5	North well 3/10/44-10/7/54
Hartford (V)	1	0.7- 16.4	4- 21	39.3- 129.4	146- 400	227- 501	346- 590	55.5 57	Well 1 3/31/50 to 1962
Troy (V)	2	0.1- 0.6	3- 11	103.1- 177.1	264- 310	396- 464	438- 563	56- 57.5	Well 1 1/25/54 to 1962
American Zinc Co.	5	9.6- 50.4	20- 119	254.5- 744.2	280- 392	521- 1080	624- 1596		Well 2 11/22/44-11/15/48
	5	15 - 23	43- 118	260.4- 422.7	236- 320	503- 638	766- 1150		Well 7 1/23/61 to 1962

Table 39. Chemical Analyses of Water in Mississippi River at Alton

(Chemical constituents in parts per million)

Date	Laboratory number	Iron (Fe)	Chloride (Cl)	Sulfate (SO_4)	Alkalinity (as $CaCO_3$)	Hardness (as $CaCO_3$)	Total dissolved minerals	Temperature (°F)	Turbidity
5/ 1/51	125197	3.6	9	54.5	120	189	230	61.5	74
6/28/51	125677	5.9	9	55.1	120	189	230	75	168
9/ 7/51	126474	4.0	15	62.7	144	203	261	73	73
9/26/51	126572	8.3	12	45.9	156	210	246	72	311
12/ 6/51	127175	3.5	10	61.7	180	257	275	45	61
1/ 3/52	127416	4.5	9	59.4	172	245	276	32	115
2/ 2/52	127720	4.6	10	71.0	160	246	279	34	96
4/29/52	128690	1.8	7	56.2	136	202	231	65	43
5/ 2/52	128746	3.6	6	52.0	136	193	224	67	84
6/ 4/52	128955	3.8	11	76.9	176	256	293	75	101
2/ 4/53	131056	0.5	15	56.0	176	224	275	41	16
3/ 6/53	131345	3.7	13	57.6	132	176	232	40	124
4/ 2/53	131622	8.3	11	71.8	144	224	266	50	220
4/29/53	131853	2.4	12	79.6	152	240	279	60	60
5/28/53	132119	4.5	14	84.1	164	248	300	75	95
7/ 2/53	132404	3.1	8	52.0	144	204	226	86	70
7/30/53	132600	1.6	36	54.6	136	224	287	88	43
10/ 1/53	133068	1.2	15	61.7	152	196	262	76	38
11/ 5/53	133314	1.4	15	53.5	164	196	249	57	33
12/31/53	133676	0.6	15	38.1	156	176	225	33	32
2/25/54	134103	1.2	17	60.3	156	208	262	46.5	43
3/31/54	134363	7.1	15	61.8	120	196	268	48	234
4/30/54	134724	3.2	16	95.0	152	256	306	68	166
6/ 2/54	134966	4.6	20	104.8	160	256	322	70.5	97
6/30/54	135189	7.6	10	44.8	128	172	206	82	293
7/28/54	135447	2.3	16	44.6	132	180	209	86	49
9/ 1/54	135693	4.3	11	43.6	152	176	230	79	155
10/ 7/54	135923	4.0	14	44.8	148	188	233	72	107
10/29/54	136135	6.5	16	83.1	144	228	284	63	143
12/ 2/54	136391	0.9	16	68.3	176	220	277	40	36
1/ 6/55	136663	3.2	19	69.1	176	228	293	39	76
3/16/55	137223	11.6	14	86.4	176	260	307	49	149
3/30/55	137321	2.2	14	64.0	160	212	257	44	45

Table 39 (Continued)

Date	Laboratory number	Iron (Fe)	Chloride (Cl)	Sulfate (SO_4)	Alkalinity (as $CaCO_3$)	Hardness (as $CaCO_3$)	Total dissolved minerals	Temperature (°F)	Turbidity
5/11/55	137675	2.0	12	78.8	180	252	301	67	47
6/ 2/55	137812	8.2	8	53.9	120	160	212	73	298
6/29/55	138071	2.3	12	65.8	160	220	276	80	38
8/ 3/55	138394	1.1	15	37.8	144	180	211	92	28
9/ 8/55	138599	1.8	8	38.1	128	172	184	79	64
10/ 4/55	138785	1.4	20	49.2	144	176	238	70	93
11/ 4/55	139004	0.5	19	57.0	140	188	272	51	20
12/ 7/55	139282	0.3	17	46.3	156	184	245	37	62
12/28/55	139420	0.6	18	47.1	172	204	264	34	20
2/10/56	139760	0.5	18	45.7	168	208	273	33	13
2/29/56	139983	1.5	23	56.7	160	228	264	42	32
3/29/56	140209	0.6	16	61.5	152	228	289	50	28
5/ 1/56	140483	7.3	17	55.5	120	172	232	53	234
6/ 1/56	140716	6.9	15	64.6	136	196	262	73	181
6/30/56	140928	2.1	13	44.2	140	168	222	84	55
8/ 3/56	141156	1.1	18	53.5	140	176	255	84	24
8/27/56	141369	3.6	14	42.4	136	164	216	80	56
10/ 1/56	141601	3.7	18	43.2	140	184	222	71	69
10/29/56	141796	1.3	19	52.7	148	180	255	62	34
11/28/56	142026	0.9	18	52.2	144	204	249	41	19
12/28/56	142239	1.3	22	53.9	160	204	289	37.5	30
1/29/57	142499	2.9	17	49.2	140	188	227	34	66
3/ 4/57	142812	1.5	21	75.5	148	212	288	42	36
3/27/57	142974	3.0	15	65.2	140	192	251	44	88
4/30/57	143298	10.	10	65.6	128	184	237	67	650
5/27/57	143484	5.2	12	75.1	144	216	295	70	129
7/ 8/57	143871	3.9	12	57.6	124	194	262	81.5	99
9/11/57	144452	3.2	18	61.1	144	200	248	76	72
10/10/57	144725	3.1	19	64.8	152	204	288	65	83
11/ 6/57	145010	2.7	17	56.8	134	202	260	52	72
11/29/57	145239	3.5	19	73.8	168	230	317	42	35
1/ 6/58	145426	2.8	15	80.8	162	232	303	35	59
1/30/58	145697	2.7	16	82.3	180	260	342	35	35
2/25/58	145869	1.0	22	81.3	186	276	346	34	22
6/10/58	147827	6.3	15	69.7	152	208	287	78	107
11/24/58	148305	2.6	19	55.31	148	188	275	50	51

Table 40. Chemical Analyses of Water in Mississippi River at Thebes, Illinois*

(Chemical constituents in parts per million)

Date	Discharge (ft/sec)	Laboratory number	Temperature (°F)	Turbidity	Iron (Fe)	Manganese	Fluoride (F)	Boron (B)	Silica (SiO_2)	Chloride (Cl)	Sulfate (SO_4)	Nitrate (NO_3)	Ammonium (NH_4)	Calcium (Ca)	Magnesium (Mg)	Sodium (Na)	Alkalinity (as $CaCO_3$)	Total hardness (as $CaCO_3$)	Total dissolved minerals
1950																			
10/18	103,000	123421	65	685	13.0	1.1			15.3	14	75.7	4.9	T	55.4	13.4	28	152	194	317
11/ 8	86,000	123582	54	259	5.0	0.6			15.8	18	119.3	3.5	0.2	65.1	18.7	33	160	240	369
1951																			
1/ 9	101,000	124122	33	102	4.0	0.3			17.8	23	102.9	7.6	T	58.4	18.9	38	160	223	360
2/15	119,000	124410		149	7.7	0.5			12.6	16	61.5	5.7	0.1	40.8	14.4	19	112	162	238
3/13	303,000	124705	38	512	15.5	1.1			16.1	8	46.3	6.1	0.0	42.7	11.6	10	112	155	223
4/15	413,000	124887	46	763	30.8	2.4			14.4	10	46.3	5.8	0.2	47.6	12.1	11	128	169	235
5/ 9		125366	58	743	28.4	1.8			23.8	9	42.6	7.8	T	46.1	10.7	13	124	160	225
6/13	410,000	125601	70	846	51.0	2.9			15.7	10	61.9	6.1	0.0	61.3	7.8	28	156	186	284
7/24	802,000	126000	81	380	13.6	0.5			16.8	7	31.5	1.0	T	41.6	6.9	13	116	133	201
9/12	344,000	126468	74	685	23.3	0.6			13.9	10	45.5	4.4	T	46.1	4.8	20	120	141	237
10/10	228,000	126667	65	306	10.6	0.9			13.5	16	73.4	5.1	T	57.4	11.4	26	156	263	299
11/15	305,000	127030	45	220	10.5	1.3			16.3	11	71.6	4.6	0.1	54.9	16.6	13	140	206	283
1952																			
1/10	156,000	127450	34	59	2.1	0.3	0.3		14.2	18	71.4	7.8	0.3	59.0	21.0	11	152	234	319
3/20	480,000	128250	47	685	25.0	1.9	0.3		12.4	8	53.9	9.3	0.1	44.6	12.3	13	116	162	213
3/12	308,000	128251	47	167	6.1	0.6	0.3		13.2	11	67.9	6.8	T	49.6	14.7	22	140	185	274
4/16	547,000	128480		500	20.0	1.8	0.2		11.0	8	76.5	6.6	0.1	50.1	14.5	16	124	185	259
6/11	198,000	129028	81	600	19.9	1.6	0.3		15.2	12	77.1	5.7	T	67.0	15.8	25	184	233	430
6/25	257,000	129099	82	372	11.1	0.9	0.3		32.0	9	79.0	6.0	T	57.2	16.5	24	164	211	318
7/30	162,000	129593		296	8.6	0.5	0.2		15.4	10	59.7	5.2	T	52.7	15.5	13	144	186	271
8/ 6	149,000	129660	81	136	5.2	0.3	0.3		18.0	12	58.4	4.3	0.0	51.2	16.3	20	156	195	278
9/10	125,000	130229	76	661	19.3	1.3			13.3	11	60.9	4.0	T	45.7	12.0	12	108	164	242
10/30	74,000	130337		136	6.8	0.3	0.3		10.9	19	109.6	2.5	T	57.9	19.2	36	160	224	351
12/ 3	84,300	130660		186	5.8	0.3			10.6	21	94.6	3.0	T	57.2	18.1	37	168	218	347
1953																			
1/19	69,500	131031	41	28	2.1	T			11.0	20	88.0	4.9	0.1	63.7	22.9	24	184	254	342
3/ 4	179,000	131346		466	14.4	0.8	0.3		10.3	14	61.7	6.2	0.1	41.5	15.0	18	116	166	242
4/ 8	337,000	131690		694	26.1	1.3	0.3		12.1	8	65.6	7.3	T	48.0	13.7	12	116	176	242
5/23	242,000	131976	46	431	15.7	0.9	0.3		9.9	10	90.5	3.9	0.1	58.5	16.1	23	152	213	310
6/ 9	170,000	132217	81	343	9.3	0.7	0.4		8.8	16	91.8	1.1	T	58.6	18.5	30	168	222	339
7/ 2	227,000	132407	85	685	52.8	2.8	0.5		14.1	10	134.3	5.4	0.0	65.6	19.0	44	180	242	414
8/ 5	128,000	132642	82	95	3.5	0.3	0.3		13.3	13	64.6	2.0	T	46.5	17.6	16	136	189	281
9/ 3	118,000	132828	80	76	1.8	0.3	0.3		9.9	15	72.7	2.8	T	48.0	16.6	29	152	188	283
10/14	67,500	133175	66	84	3.6	0.2	0.2		6.4	18	118.9	2.0	0.1	58.1	19.3	43	168	225	383
11/10	70,000	133391	72	91	3.7	0.3	0.3		7.0	17	111.1	1.7	T	57.5	19.1	40	168	223	384
12/ 9	71,100	133607	46	14	2.3	T	0.3		7.1	21	74.9	3.5	0.1	55.5	21.4	29	180	227	333
1954																			
2/10	52,800	133982	48	27	2.2	0.1	0.3		11.4	26	91.9	6.2	T	64.2	20.7	44	204	246	403
3/10	79,000	134192	48	167	6.4	0.5	0.5		9.9	20	97.0	3.8	0.4	58.0	17.8	37	168	219	365
4/21	139,000	134673	49	455	11.0	0.5	0.4		8.8	14	86.4	6.9	T	54.8	21.0	18	148	224	325
5/ 4	211,000	134888	52	1240	11.0	1.4	0.2		7.6	12	85.5	6.9	T	57.0	17.8	20	148	216	317
6/16	194,000	135070	72	1200	25.0	1.3		T	10.1	12	71.5	8.4	T	55	3.1	48	156	150	308
7/ 7	239,000	135263	82	750	T	0.9	0.4	0.2	12.6	6	45.7	8.2	T	47.2	19.0	0	132	196	238
8/10	114,000	135635	78	102	T	0.3	0.4	0.3	9.2	15	112.3	2.8	T	50.9	17.1	38	140	198	348
9/22	120,000	135489	66	263	1.4	0.1	0.7	0.0	6.5	12	71.2	3.1	T	48.6	15.4	20	136	185	269
10/ 4	113,000	136203	48	119	T	0.3	0.3	0.0	10.9	11	75.9	4.2	T	52.3	17.9	10	128	204	274
12/ 7	92,000	136518	41	13	0.1	0.3	0.3	0.0	20.0	16	71.6	1.8	T	55.8	19.2	12	148	219	284
1955																			
2/ 8	69,900	136898	42	44	2.8	0.1	0.3	0.0	11.6	17	80.9	4.2	0.0	66.5	23.1	12	176	262	323
5/10	137,000	137641	70	145	5.0	0.4	0.1	0.0	8.6	9	90.3	5.0	T	58.3	15.1	29	160	208	312
6/15	177,000	137915	62	500	8.6	1.2	0.2	0.0	7.0	14	80.8	3.1	T	46.4	18.5	24	140	192	285
7/ 7	129,000	138105	71	950	18.0	1.6	0.1	0.0	9.6	15	64.0	3.4	0.2	50.9	15.3	16	136	191	262
8/ 4	82,900	138357	81	112	7.0	0.4	0.3		14.4	16	111.5	5.1	T	53.8	18.6	35	144	211	354
9/ 2	109,000	138606	67	60	2.0	0.2	0.2	0.1	7.4	17	101.2	3.8	T	51.4	19.2	33	148	208	322
10/ 5	103,000	138779	62	201	1.1	T	0.2	0.1	7.2	19	95.2	4.1	T	46.9	16.3	37	136	185	318
11/ 2	82,100	139003	52	84	3.4	0.4	0.2	0.0	7.2	19	105.5	4.5	T	58.8	16.8	32	144	216	335
12/ 1	52,900	139211	44	45	1.9	0.1	0.2	0.4	7.4	22	79.4	5.4	T	54.8	18.3	34	168	212	331
1956																			
2/15	69,700	139892		600	19.0	1.9	0.2	0.2	9.5	20	66.4	4.2	0.8	43.0	17.1	34	152	178	291
2/29	88,200	139960	41	242	7.9	0.7	0.3	0.0	9.5	24	80.0	7.7	T	53.4	17.8	26	140	207	323
4/14	113,000	140301	49	86	3.8	0.5	0.3	0.1	5.4	15	75.3	4.7	0.1	48.0	18.3	26	148	195	276
5/ 1	188,000	140545	52	80	2.8	0.5	0.2	0.0	11.1	13	66.4	4.4	0.1	46.5	15.1	13	120	179	302
6/ 1	144,000	140739	69	119	6.3	0.5	0.2	0.0	6.7	16	90.1	5.2	0.1	51.5	17.7	25	136	202	296
7/ 6	146,000	140981	71	198	8.2	0.2	0.1	0.0	7.5	14	89.1	3.6	0.1	47.3	16.9	27	132	188	302

*From Larson and Larson (1957)

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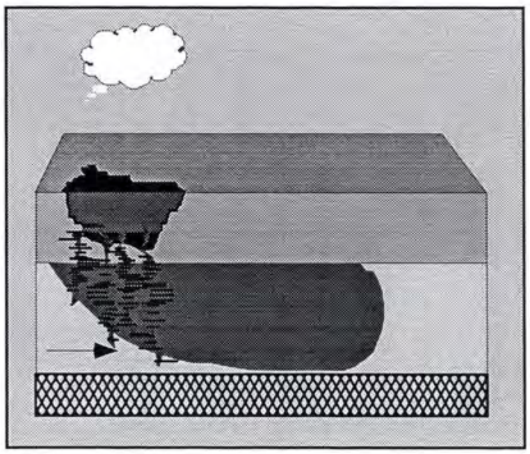
SOURCE EVALUATION STUDY

Sauget Area 1

Sauget and Cahokia, Illinois



Submitted to
Solutia Inc.

May 21, 2001

Groundwater Services, Inc.
2211 Norfolk, Suite 1000, Houston, Texas 77098

EXECUTIVE SUMMARY

An evaluation of the source of impacted groundwater at Site I within Sauget Area 1 was performed to evaluate two questions:

1) *What is the dominant source mechanism at the site?*
2) *What is the effect of an intensive pump-and-treat system on the lifetime of the source?*

Dominant Source Mechanism

Two source mechanisms that have the potential to be active at the Sauget site are: 1) *leaching* of unsaturated source materials, and 2) residual Dense Nonaqueous Phase Liquid *(DNAPL) dissolution* (see Figure 1). Six general indicators were evaluated to assess which of these two source mechanisms are primarily responsible for the observed plumes associated with Site I.

The analysis indicated that *DNAPL dissolution* is a major source mechanism at Site I based on an analysis of six different indicators. The following indicators support the conclusion that trapped residual DNAPL is present: dissolved constituent concentrations increase with depth, site constituents extend from the water table to the bottom of the water-bearing interval, and three constituents are found at concentrations that suggest the possible presence of non-mobile residual DNAPL. Some leaching of unsaturated waste/soil materials may also be occurring, as some constituents did not show increasing concentrations with depth. Overall, however, **DNAPL dissolution** appears to be the dominant source mechanism at Site I.

Source Conceptual Model

If DNAPL dissolution is the dominant source mechanism at Site I, it is likely that the DNAPL in the unconsolidated valley fill deposits is present as thin vertical fingers and small horizontal pools throughout the entire vertical extent of the water-bearing unit. Only a small fraction of the total DNAPL mass can ever be removed by pumping any "free-phase" DNAPL pools, if they are found. The rest of the DNAPL is immobile, and will serve as a long-term continuing source of constituents to groundwater.

The current natural mass removal rate via dissolution from the Site I source zone was estimated to be 7000 kg/yr assuming uniform source concentrations throughout the source zone.

Effect of Pumping

As shown by DNAPL dissolution expressions, increasing the flow rate through a DNAPL source zone will significantly decrease the concentration of constituents in the extracted groundwater. For example, if the flowrate through a DNAPL source zone is increased by a factor of 8.9 (to 1500 gpm) due to intensive pumping, the resulting concentration is likely to decrease by a factor of 3.6 while pumping is active, resulting in an overall increase in the mass removal rate of only 2.5 times. Therefore, an intensive pump-and-treat system at Site I with 8.9 times the natural flowrate through the source area (an achievable pumping rate if there is no reinjection) would result in an initial mass removal rate of 17,500 kg/yr.

A planning level source lifetime calculation was done to estimate the relative performance of various remediation schemes. This analysis, while not providing high-confidence estimates of the absolute time to cleanup, does indicate that with an assumed mass of 410,000 kg of VOCs + SVOCs in the saturated zone below Site I, intensive pumping over a 10 to 30 year period does not appear to have an appreciable effect on overall source lifetime (i.e., ≤ 10% reduction). Similar limitations are expected for Sites G/H/L as well.

INTRODUCTION

As requested by Solutia Inc. (Solutia), Groundwater Services, Inc. (GSI), has completed a study of hydrogeologic, source, and fate and transport data from the Sauget Area 1 located in Sauget and Cahokia, Illinois. The study was conducted to: 1) help determine what type of source mechanisms are responsible for dissolved constituents found in the affected groundwater, and 2) determine the feasibility of remediating this source area by aggressive pumping. This letter report summarizes the results of the study.

PROJECT BACKGROUND

An extensive RI/FS study of Sauget Area 1 is now being conducted by Solutia. Data from two groundwater monitoring well transects indicates the presence of dissolved constituents migrating west in groundwater from the vicinity of one of the six source areas in Area 1 (i.e., Site I) at concentrations exceeding Illinois Class II groundwater standards.

Source Site I

Site I originally was a sand and gravel pit which received industrial and municipal wastes from 1931 to 1957. Site I is approximately 19 acres in area and underlies a large, fenced, controlled-access, gravel covered truck parking lot and the Sauget City Hall and associated parking lots (Sauget Area 1 EE/CA and RI/FS Support Sampling Plan). Soil samples collected from Site I have indicated elevated levels of volatile organic compounds (e.g., benzene, chlorobenzene); semi-volatile organic compounds (e.g., naphthalene, trichlorobenzene); pesticides; herbicides; PCBs; and metals.

Hydrogeology

Sauget Area 1 is located in the Mississippi River floodplain in an area referred to as the American Bottoms. The geology of the area is described as consisting of unconsolidated valley fill deposits (Cahokia Alluvium) overlying glacial outwash material (Henry Formation). In general, the permeability of the unconsolidated material increases with depth, with the outwash material being comprised of medium- to coarse-grained sand and gravel. The hydrogeologic conceptual model divides the unconsolidated water-bearing unit into three horizons: the shallow horizon (generally 15-30 ft deep), the middle horizon (generally 30-70 ft deep), and the deep horizon (generally 70-110 ft deep).

These unconsolidated deposits are underlain by limestone and dolomite bedrock.

Study Constituents

For this study, two classes of constituents were evaluated. The two constituent classes were selected based on prevalence and concentration in groundwater, and include:

- Volatile Organic Compounds (chlorinated and non-chlorinated), and
- Semi-Volatile Organic Compounds (chlorinated and non-chlorinated).

SOURCE MECHANISMS

Knowledge of which source mechanisms are active at a site is important for developing an accurate conceptual model of constituent fate and transport, and for developing appropriate remedial responses. Two source mechanisms that have the potential to be active at the Sauget site are leaching of unsaturated source materials and residual DNAPL dissolution (see Figure 1).

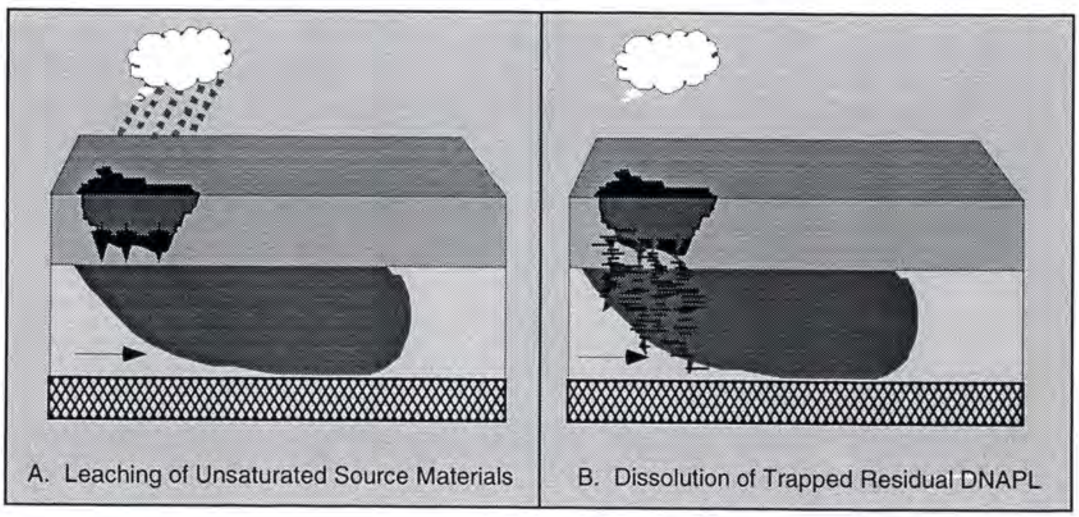


FIGURE 1. Two Potential Groundwater Source Mechanisms

Leaching of unsaturated source materials (see Panel A in Figure 1) results from infiltration of rainfall through near-surface source materials such as waste materials in the source areas and contaminated unsaturated soils. Residual DNAPL dissolution (see Panel B in Figure 1) occurs when soluble organic constituents dissolve from trapped residual DNAPL fingers and pools that entered the subsurface when the source area was active.

EVIDENCE OF SOURCE MECHANISMS

The available groundwater data were evaluated to help assess the likelihood that the two most likely source mechanisms are present at the site. The following six indicators were used:

General Indicators of Strong DNAPL Dissolution Processes:

- Indicator 1: Concentrations are generally increasing with depth.
- Indicator 2: Constituents are found deep in the water-bearing unit.
- Indicator 3: Concentrations are above 1% of the pure-phase solubility.
- Indicator 4: Results of EPA Quick Reference Fact Sheet "Estimating Potential for Occurrence of DNAPL at Superfund Sites," (Newell and Ross, 1992).

General Indicators of Strong Soil Leaching Processes:

- Indicator 5: Leachate concentrations (as indicated from TCLP tests of unsaturated waste materials) are greater than groundwater concentrations in the shallow horizon.
- Indicator 6: Concentrations in the shallowest horizon are greater than in deeper horizons.

To assess these indicators, groundwater constituent data were compiled (see Tables 1-2). Data from the groundwater transect (well AA-I-S1 for Site I) were used to evaluate constituent concentrations in the shallow horizon (< 30 ft deep) vs. middle horizon (30-70 ft deep) vs. deep horizon (70-110 ft deep) (see Table 1). Note that only the transect monitoring well closest to the source area was evaluated.

RESULTS

<u>Indicator 1:</u> An evaluation of groundwater data for Site I shows that the sum of *maximum* detectable VOCs + SVOCs in groundwater concentrations from the deep horizon is 47.5 mg/L, compared to only 22.1 mg/L in the shallow horizon (see Table 1). This trend is also seen in the majority of the individual VOC and SVOC constituents. For example, the maximum chlorobenzene concentration increases from 8.7 mg/L in the shallow horizon, to 20 mg/L in the middle horizon, and to 34 mg/L in the deep horizon. Of the five constituents with maximum concentrations greater than 1 mg/L, three (chlorobenzene, 1,2,4-trichlorobenzene, and 1,4-dichlorobenzene) have their maximum

concentrations in the middle or deep horizon. The other two, cis/trans 1,2-dichloroethene and 4-chloroaniline, have the maximum concentration in the shallow horizon.

Indicator 2: Site constituents are found throughout the entire depth of the unconsolidated unit, from the water table surface to locations over 100 ft deep.

Indicator 3: Three site constituents (chlorobenzene, 1,4 dichlorobenzene, and fluoranthene) are found in concentrations that exceed 1% of each respective pure-phase solubility (see Table 1).

Indicator 4: Based on site historical data and observed groundwater concentrations, the EPA Fact Sheet "Estimating Potential for Occurrence of DNAPL at Superfund Sites," shows a "**High-Moderate**" Potential for DNAPL at Site I.

Indicator 5: Of the six constituents where a comparison could be made, five had higher concentrations in the groundwater than in the leachate from waste materials, suggesting that leaching was not responsible for the highest groundwater concentrations at Site I (see Table 2).

Indicator 6: As described above, only two of the five constituents with concentrations greater than 1 mg/L (cis/trans 1,2-dichloroethene and 4-chloroaniline) have their maximum concentrations in the shallow horizon.

KEY POINT: SITE I SOURCE MECHANISMS

DNAPL dissolution is a major source mechanism at Site I based on an analysis of the evaluated indicators. Dissolved constituent concentrations increase with depth, site constituents extend from the water table to the bottom of the water-bearing interval, and three constituents are found at concentrations that suggest the possible presence of non-mobile residual DNAPL.

Some leaching of unsaturated waste/soil materials may also be occurring, as some constituents such as 4-chloroaniline did not show increasing concentrations with depth. Overall, however, DNAPL dissolution appears to be the dominant source mechanism at Site I.

POTENTIAL FOR SOURCE REMEDIATION

Conceptual Model of Source

The following discussion summarizes our conceptual model of the DNAPL source located in the saturated zone beneath Site I:

- DNAPL is present as "fingers" and "pools" in the saturated zone extending from approximately 15 to 110 ft below the surface (see Figure 2 for a conceptual figure).

Supporting Information: "Once penetration of the capillary fringe occurs, downward movement will continue until all the CHC (chlorinated hydrocarbon) solvent is present as suspended fingers (ganglia) in the porous media and/or as pools of CHC perched on low-permeability zones. Once a pool starts to form on top of a low-permeability layer somewhere above the bottom of the aquifer, a continued supply of CHC will cause (1) enlargement of the pool, (2) penetration of the layer, and/or (3) spawning of new downward-moving fingers at the perimeter of the layer." Johnson and Pankow (1992)

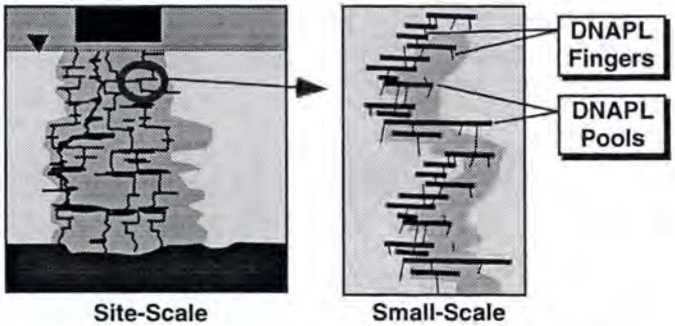


FIGURE 2. Conceptual Model of DNAPL Source Zone

- Small horizontal pools of DNAPL are present throughout the entire vertical extent of the saturated zone, and not just at the bottom of the unit.

Supporting Information: "In granular aquifers, small horizontal zones of residual or free-phase DNAPL need not be caused by particularly low permeability zones such as silt or clay. A minor contrast in grain size distribution and hence permeability, as from a coarse sand layer to a fine sand, causes variation in DNAPL entry pressure. A DNAPL will accumulate on the finer-grained layer while spreading laterally until it reaches the edge of the layer, or until the height of the free-product accumulation on the layer exceeds the entry pressure for the layer." Pankow and Cherry (1996)

- Much or most of the DNAPL mass is present in the trapped residual state that cannot be recovered by pumping.

Supporting Information: "Note that after the continuous NAPL body has been converted to a residual form, the individual NAPL blobs are held very tightly in the porous media by capillary forces. Wilson and Conrad (1984) evaluated the force required to mobilize and completely sweep away residual blobs in porous media in terms of the hydraulic gradient a pumping system would have to generate to either 1) begin blob mobilization, or 2) mobilize all blobs in a porous medium. This relationship, presented as a graph of hydraulic conductivity vs. required hydraulic gradient, indicates that mobilization of NAPL blobs by pumping will occur only in very coarse porous media with a very high hydraulic gradient.

The rest of the blobs will stay trapped in the porous media, serving as a long-term source of dissolved contaminants." (Wiedemeier et al., 1999)

(Note: For the unconsolidated alluvial fill deposits at this site, Wilson and Conrad's analysis indicates that a gradient of 0.5 ft/ft would be required to begin to mobilize NAPL blobs. This is 500 times the current hydraulic gradient, and impossible to effect over the entire source area without extensive pumping and re-injection).

- It is extremely unlikely that any DNAPL that may be present at the site is still mobile or will become mobile under current conditions.

  Supporting Information: "Once the release of DNAPL into the subsurface ceases, subsurface movement of DNAPL also ceases soon thereafter, perhaps within weeks or months at solvent sites. The resulting immobile DNAPL then exists in the DNAPL source zone as "residual" non-aqueous liquid and also possibly as "free-product" accumulations ponded on lower permeability layers within aquifers, or on the tops of aquitards. The free-product DNAPL will not become mobile again unless a release of more DNAPL causes further accumulation in the same zones, or unless there are changes in pressure in the surrounding water phase due to groundwater pumping or injection." Pankow and Cherry (1996)

- The presence of pools that can be pumped is relatively rare at DNAPL sites, and if such pools are found and pumped, only a small fraction of the total DNAPL in place is removed.

  Supporting Information: "In field investigations of sites where extensive solvent contamination exists, pools of free-product solvent are only rarely found, even when their existence is not in doubt." "It is the author's experience that chlorinated solvents with their high densities form thick pools only rarely." Pankow and Cherry (1996)

  "Therefore, in a practical sense, NAPL removal translates to recovery of a small percentage of NAPL at a site (i.e., whatever continuous NAPL can be collected)." Wiedemeier et al., (1999)

- The presence of DNAPL pools and fingers will only occupy a small fraction of the available pore space in the source zone.

  Supporting Information: "However, heterogeneity has a marked influence on the direction of DNAPL migration. A random distribution of permeability and displacement pressure will result in a highly erratic pattern of DNAPL flow..." "The remarkable sensitivity of DNAPL penetration to the capillary-hydraulic properties can be expected to result in highly complex, seemingly chaotic saturation distributions in the subsurface." "Even in the idealized case of a perfectly homogeneous medium, DNAPL can be expected to penetrate in the form of narrow, elongated distributions in which the mean saturation of DNAPL is small." Pankow and Cherry (1996)

- The source will persist for a long time.

  Supporting Information: "These calculations suggest that zones of residual DNAPL and especially pool DNAPL can persist in the subsurface and contribute to groundwater contamination for decades to centuries." "For most chlorinated solvents, the rate of *dissolution of pools will be sufficiently slow that the DNAPL source zones will cause significant contamination of the groundwater for centuries or more." Pankow and Cherry (1996)*

- Pumping can increase the rate that mass is removed from the source, but the removal efficiency will be much less than the removal efficiency for natural attenuation.

  Supporting Information: "However, the increase in mass removal (by pumping) will not be in proportion to the increase in the groundwater pumping rate because of limitations on the DNAPL dissolution kinetics, and because of further dilution with clean water from outside the source zone." Pankow and Cherry (1996)

KEY POINT: CONCEPTUAL MODEL FOR SOURCE

Most of the Site I DNAPL in the unconsolidated valley fill deposits is present as thin vertical fingers and small horizontal pools throughout the entire vertical extent of the water-bearing unit. Only a small fraction of the total DNAPL mass can ever be removed by pumping free-phase pools, if they are found. Under current conditions, the rest of the DNAPL is immobile, and will serve as a long-term continuing source of constituents to groundwater.

ESTIMATED NATURAL DISSOLUTION RATE

Natural Groundwater Flushing Rate

Separate hydraulic conductivity and hydraulic gradient data were developed for the shallow, middle, and deep horizons of the unconsolidated deposits.

The hydraulic conductivity estimates developed for the model were based on: 1) literature reports, and 2) preliminary analysis of RI/FS slug test data. The literature reference (Ritchey and Schicht, 1982) reported that the hydraulic conductivity for the unconsolidated material used for water supply in the American Bottoms area ranged from $5x10^{-2}$ to $1.4x10^{-1}$ cm/sec.

The analysis of RI/FS slug test data from Site I wells showed the following hydraulic conductivities:

Horizon	Site I (well ST-I-S) (cm/sec)
Shallow	$4.5x10^{-3}$
Middle	$5.1x10^{-2}$
Deep	$1.3x10^{-1}$

Using the data from the literature report, slug test results, and calibration work, the following hydraulic conductivities were used in the model:

- Shallow Horizon: $1x10^{-2}$ cm/sec
- Middle Horizon: $1x10^{-1}$ cm/sec
- Deep Horizon: $1x10^{-1}$ cm/sec

Using RI/FS potentiometric surface maps provided by Roux Associates, Inc., the following hydraulic gradients were used in the model:

- Shallow Horizon: 0.001 ft/ft
- Middle Horizon: 0.001 ft/ft
- Deep Horizon: 0.001 ft/ft

These values yield the following representative values for groundwater Darcy velocity at the site:

- Shallow Horizon: 10.4 ft/yr
- Middle Horizon: 104 ft/yr
- Deep Horizon: 104 ft/yr

As shown by the data, the shallow horizon of the unconsolidated deposits is less permeable, and has a much lower groundwater velocity than the more coarse-grained middle and deep horizons.

The hydrogeologic conceptual model divides the unconsolidated water-bearing unit into three horizons: the shallow horizon (generally 15-30 ft deep), the middle horizon (generally 30-70 ft deep), and the deep horizon (generally 70-110 ft deep). Therefore the assumed saturated thicknesses for the shallow, middle, and deep units were: 15 ft, 40 ft, and 40 ft, respectively. When a 1400 ft wide source zone is assumed (the width of Site I perpendicular to groundwater flow), a naturally-occurring groundwater flushing rate of 168 gpm is obtained (3.1 gpm for the shallow unit, 82.5 gpm for the middle unit, and 82.5 gpm for the deep unit).

Natural Mass Removal Rate

The average total VOC + SVOC concentrations from the transect well closest to Site I (well AA-I-S1) are 13.3 mg/L, 21.9, mg/L, and 19.9 mg/L for the shallow, middle, and deep horizons, respectively. For this planning-level calculation, it was assumed that these concentrations extended throughout the entire width of the Site I source zone, a potential overestimation (however, if all other source removal calculations use the same assumptions, the relative results will be accurate). Therefore, the mass removal rate under natural conditions was estimated by multiplying average VOC + SVOC concentrations for each horizon by the flow for each horizon, and converting to a mass rate of kg/yr leaving the source zone (3.78 L/gal; 1440 min/day; 365 day/yr; 10^{-6} kg/mg). This calculation resulted in the following naturally-occurring mass removal rate totaling approximately 7000 kg/yr from all three horizons:

• Shallow Horizon:	82 kg/yr
• Middle Horizon:	3,613 kg/yr
• Deep Horizon:	3,271 kg/yr
TOTAL:	6,966 kg/yr

KEY POINT: NATURAL MASS REMOVAL RATE

The natural mass removal rate from the Site I source zone is estimated to be 7000 kg/yr assuming uniform source concentrations throughout the source zone.

Assumed Flowrate From An Intensive Pump-and-Treat System

Three methods were evaluated to provide a planning-level estimate of the flowrate from an intensive pump-and-treat system at Site I (see Appendix A). First, an empirical well yield relationship (Driscoll, 1986) based on transmissivity, expected drawdown, and assumptions for other variables in the nonequilibrium (Jacob) equation was used. The second method was based on typical well yields from regional water supply wells as reported by Schicht (1965):

> "It is a general practice of industries and municipalities to place a well in operation and pump it at high rates, often about 1000 gpm."

The third method was based on evaluating specific capacity (well yield divided by drawdown) provided by Schicht (1965).

These calculation approaches suggest that an intensive pumping system for Site I could yield 1000–2500 gpm. For the purpose of this project, a value of 1500 gpm was used.

KEY POINT: GROUNDWATER FLOWRATE FROM INTENSIVE PUMPING

An intensive pump-and-treat system was assumed to have a yield of 1500 gpm.

EFFECT OF PUMPING GROUNDWATER

Because most of the DNAPL is trapped and cannot be removed by direct pumping, a groundwater pump-and-treat system will generally not remove DNAPL directly, but instead will slowly dissolve the DNAPL trapped in fingers and pools. While this dissolution process is relatively slow and inefficient, it will remove DNAPL mass.

Dissolution Kinetics for DNAPL Fingers and Pools

Several analyses have been performed to evaluate the effect of increased pumping rates on the DNAPL dissolution rate for both fingers and pools. In a key paper written by Hunt et al. in 1988, the authors developed relationships for the kinetics of dissolution in NAPL source zones. They evaluated laboratory studies and mass transfer approaches used in the chemical engineering literature, and derived dissolution expressions for residual NAPL ganglia (also called "fingers" or "blobs"). They concluded that:

"Ganglion lifetimes are weakly dependent on flow velocity such that to decrease the lifetime from 100 years to 10 years requires a three order of magnitude increase (x1000) in flow velocity."

In other words, increasing the groundwater pumping rate will increase the finger dissolution rate, but only slightly based on this relationship:

$$\frac{\text{mass transfer rate with pumping}}{\text{mass transfer rate without pumping}} = 10^{\frac{\log 10\left[\frac{Q_{\text{pumping}}}{Q_{\text{natural}}}\right]}{3}}$$

Using this NAPL dissolution relationships reported by Hunt et al. (1988), a 1500 gpm pumping system (a 8.9 times increase in the natural flow rate through the system) would result in a 8.9 fold *increase* in water flushed through the system, but a 4.3 fold *decrease* in effluent concentrations, resulting in a net increase in mass removed only by a factor of 2.1:

$$\frac{\text{mass transfer rate with pumping}}{\text{mass transfer rate without pumping}} = 10^{\frac{\log 10\left[\frac{1500\text{ gpm}}{168\text{ gpm}}\right]}{3}} = 2.1$$

The same type of concentration reduction is expected when higher groundwater flowrates are used to dissolve NAPL pools. Dissolution kinetic relationships developed by Johnson and Pankow (1992) indicate that the mass transfer rate (and pool lifetime) changes with the square root of groundwater velocity:

$$\text{Pool Dissolution Time (yrs)} = 2.43\text{x}10^{-5}\ \rho\ C_{sat}\ [\ l_p^3/D_v\ v_d\ ]^{0.5}$$

where:

ρ	=	DNAPL density (g/m^3)
C_{sat}	=	saturation concentration (g/m^3)
l_p	=	length of pool in direction of groundwater flow (m)
D_v	=	vertical dispersion coefficient (m^2/s)
v_d	=	Darcy velocity for groundwater (m/day)

Therefore, increasing the groundwater flowrate over a pool by a factor of 8.9 would result in an initial concentration *decrease* by a factor of 3.0 (approximately the square root of 8.9), and the overall *increase* in the mass removal rate by only a factor of 3.0.

Note that these theoretical expressions are supported by lab and field data (e.g., see Pankow and Cherry, 1996). Because source zones include a mixture of pools and fingers, it was assumed in this study that increasing the groundwater flowrate through the source zone by a factor of 8.9 (by pumping) would increase the mass transfer by a factor of 2.5 (the mid-point of finger value of 2.1 and pool values 3.0) when pumping was started. This is because groundwater concentrations decrease by a factor of 3.6 due to mass transfer effects. Note that after pumping is stopped, the concentrations would rebound and increase by a factor by the same amount (in the case of these calculations, by a factor of 3.6).

KEY POINT: EFFECT OF PUMPING GROUNDWATER ON CONCENTRATIONS

As shown by DNAPL dissolution expressions, the mass removal rate from a DNAPL source zone is only weakly dependent on the groundwater pumping rate. For example, if the flowrate though a DNAPL source zone is increased by a factor of 8.9 due to intensive pumping, the mass removal rate will only increase by a factor of 2.5 (a representative value for effects of pumping on DNAPL finger and DNAPL pool dissolution) because concentrations in the recovered groundwater would be reduced by a factor of 3.6 due to mass transfer effects.

Mass Removal Rate of Intensive Pump-and-Treat System

Under an intensive pumping scenario with an increase in natural flow (from 168 gpm to approximately 1500 gpm), the groundwater concentrations being removed from the source are expected to fall to between one-third to one-fifth of the observed concentrations under lower flow, natural conditions. Assuming a middle value of post-pumping concentrations that are 3.6 times smaller than the natural concentrations, the initial VOC + SVOC effluent concentrations from an intensive groundwater pump-and-treat system are estimated to be: 3.7 mg/L, 6.0 mg/L, and 5.5 mg/L for the shallow, middle, and deep units, respectively.

Therefore, under an intensive pump-and-treat scenario where 1500 gpm are being flushed through the Site I source zone (an 8.9-fold increase in the flushing rate), the initial mass removal rate is predicted to only increase by a factor of 2.5, from 7000 kg/yr to 17,500 kg/yr due to mass transfer effects related to DNAPL dissolution. Note that this is only the initial mass removal rate for the intensive pumping case, and that this concentration will drop slowly over time as mass is removed from the system.

KEY POINT: MASS REMOVAL RATE FROM INTENSIVE PUMPING

An intensive pump-and-treat system was estimated to have an initial mass removal rate of 17,500 kg/yr, accounting for both the increased flowrate through the system and decreased concentrations in groundwater.

Effect of Pumping on Source Lifetime

Estimating source longevity is a process involving considerable uncertainty, as the original mass in place, mass removal rate, and the change in the mass removal rate over time must all be known. While absolute estimates have a high level of uncertainty, the relative comparison of remediation alternatives can be made with more confidence. In the analysis below, the absolute values for source lifetime should be considered highly uncertain, while the relative comparisons should be considered more accurate.

<u>Estimated Source Mass</u>

A range of estimates of source mass were developed, assuming that the entire saturated zone below Site I is affected by DNAPL. Then the calculated mass removal rates for natural attenuation and an intensive pump-and-treat system were used to estimate source longevity.

Source mass is a function of source volume, the porosity, the residual saturation of DNAPL in the source zone, and the fraction of source volume containing DNAPL. At Site I, the estimated source volume is 1400 ft by 95 ft by 500 ft, or 66,500,000 ft^3. Residual saturation (the fraction of open pore space occupied by DNAPL) values are typically assumed to be between 0.01 and 0.15

(see Pankow and Cherry, 1996), and a value of 0.05 was used for this analysis. A porosity of 0.35 was considered representative of the unconsolidated alluvial deposits at the site. Finally, it was assumed that 1% of the aquifer volume contains residual DNAPL.

Based on these assumptions, a planning-level estimate for the volume of DNAPL under Site I was estimated to be 87,000 gallons. Assuming an average density of 1.25 (based on an average of the density of chlorobenzene and 1,4-dichlorobenzene, two of the most commonly-found site constituents), the estimated mass of DNAPL is approximately 410,000 kg. Note that the actual mass may be more or less, but for the purpose of performing relative calculations of source longevity this value appeared to provide reasonable results.

KEY POINT: SITE I SOURCE MASS ESTIMATE

A planning-level source mass estimate of 410,000 kg of VOCs+SVOCs was estimated for the DNAPL source zone below Site I. There is considerable uncertainty in this estimate, with the actual mass potentially being higher or lower than 410,000 kg.

Source Decay Model

A simple source model, originally developed as part of the BIOSCREEN model (Newell et. al. 1996, EPA/600/R-96/087) and now being included as part of the BIOCHLOR model (Aziz et al., 2000, EPA/600/R-00/008) was used to estimate the lifetime of the groundwater source at Site I under different remediation options.

In this simple box model, the source zone is considered to be located in a box containing some mass of dissolvable contaminants. The rate at which contaminants leave the box is estimated from the rate at which flowing groundwater removes contaminants from the box. The time required to achieve a cleanup standard can then be estimated by comparing the mass of contaminants in the box vs. the time required to remove contaminants from the box. To more closely match real-site conditions, the source concentration is assumed to decay over time, in proportion to the remaining source mass (Wiedemeier et al., 1999). With this assumption, the source concentration over time can be described using:

$$C_t = C_{so} \exp^{(-k_s t)}$$

where:

C_t	=	Source concentration at time t (mg/L)
C_{so}	=	Observed source concentration at t = 0 (mg/L)
t	=	Time (years)
k_s	=	Source decay coefficient (1/year)

(Note that this decay coefficient **is not** related in any way to first-order decay coefficients reported in the literature for natural attenuation, as the literature values typically represent decay half-lives from 0.1 to 10 years and represent biodegradation of dissolved contaminants in the plume *once they have left the source.* The source decay coefficient values represent how quickly a source zone is being depleted, and will usually have much longer half-lives, typically tens or hundreds of years.)

The source decay coefficient, representing how quickly the source is being depleted, can be derived using estimates of the source mass and rate that contaminants leave the source (Newell et al., 1996):

$$k_s = \frac{Q \cdot C_{so}}{M_o}$$

where:

Q	=	Groundwater flowrate through source zone (L/year)
C_{so}	=	Observed source concentration at time = 0 (mg/L)(or kg/L)
M_0	=	Dissolvable mass in source at time = 0 (mg)(or kg)

This model assumes that the only mass leaving the source zone is dissolved in the water flowing through the source zone. Note that Q and C_{so} are related; the thickness of the source zone should be matched with an appropriate average concentration for that entire depth horizon.

With a first-order source decay term, the source concentration at any time can be derived, providing the time required to reach any concentration:

$$t = -\frac{1}{k_s} \ln\left(\frac{C_t}{C_{so}}\right)$$

where:

t	=	Time required to reach concentration C_t (years)

Five Source Lifetime Cases

For this analysis, five different cases were evaluated using the source lifetime described above:

Case 1: Natural attenuation only (initial removal rate of 7000 kg/yr)

Case 2: 1 year of intensive pump-and-treat (initial removal rate of 17,500 kg/yr), followed by natural attenuation

Case 3: 5 years of intensive pump-and-treat (initial removal rate of 17,500 kg/yr), followed by natural attenuation

Case 4: 10 years of intensive pump-and-treat (initial removal rate of 17,500 kg/yr), followed by natural attenuation

Case 5: 30 years of intensive pump-and-treat (initial removal rate of 17,500 kg/yr), followed by natural attenuation

With this approach (see Appendix B), the following times to cleanup were estimated:

		Estimated Time to Cleanup (years)	% Reduction from Natural Attenuation Only
Case 1	Natural Attenuation Only	488	-
Case 2	1 Yr of Intensive Pump-and-Treat + Natural Attenuation	486	0.4% reduction
Case 3	5 Yrs of Intensive Pump-and-Treat + Natural Attenuation	480	2% reduction
Case 4	10 Yrs of Intensive Pump-and-Treat + Natural Attenuation	472	3% reduction
Case 5	30 Yrs of Intensive Pump-and-Treat + Natural Attenuation	441	10% reduction

Figure 3 shows a comparison of source concentrations vs. time for two of the six cases.

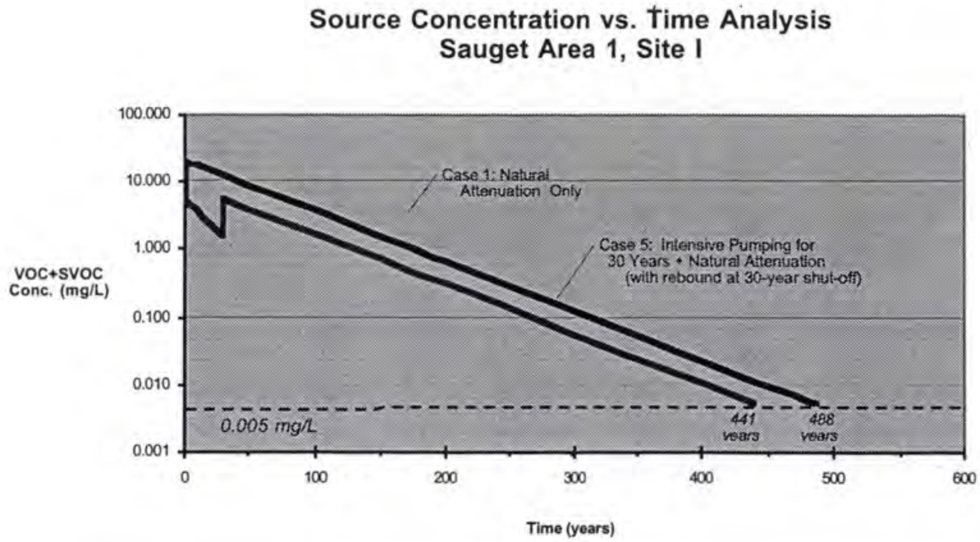


FIGURE 3. Source concentration vs. time graphs for Case 1 (Natural Attenuation Only) and Case 5 (30 Years of Intensive Pump-and-Treat + Natural Attenuation). For Case 1, concentrations start at 20 mg/L and decline as a first order decay relationship over time. For Case 5, the source concentration starts at 20 mg/L, but concentrations are reduced by a factor of 3.6 due to mass transfer effects caused by the almost 9 times increase in groundwater flow through the source zone. After 30 years, pumping is stopped, groundwater flow is restored to natural conditions, and mass transfer effects cause an increase in concentration by a factor of 3.6 (the "rebound" effect). Overall, the source modeling exercise shows that with the source assumptions described in the text, the time required to restore groundwater is reduced only slightly by 30 years of intensive pumping, from 488 years to 441 years (10% reduction).

Sensitivity Analysis

The source lifetime analysis has several areas of uncertainty, and should be used to evaluate relative differences between remediation alternatives rather than to provide an absolute source lifetime estimate. Significant sources of uncertainty include:

- *The assumption that concentrations observed in well AA-I-S1 extend throughout the entire 1400 ft source width of Site I.* If some sections of the 1400 ft source width of Site I are lower concentration, the following impact on the source lifetime is expected: 1) for the natural attenuation case, the overall source lifetime estimates will not change as both the removal rate and the mass in the source are functions of the source width; and 2) for the pumping case, some reduction in overall source lifetime is expected as source mass is dependent on source width but removal rate is not (it is dependent on pumping rate and expected concentrations).

- *The assumptions that the source is represented by a residual DNAPL saturation of 0.05 and that 1% of the source zone is impacted by DNAPL residual.* These assumptions have a great deal of uncertainty (the literature reports that residual saturations can be as high as 0.50), and were selected in part to yield source lifetimes in the range of several hundreds of years to match the source conceptual model discussed above. If the source is much smaller than the estimated 410,000 kg of VOCs+SVOCs, then the impact of a pumping system will be greater, and greater than a 1% to 10% reduction in source lifetime will be realized. If the source mass is only 41,000 kg (an unlikely event based on the persistence of the source to date), then an intensive pump-and-treat system is predicted to reduce the source lifetime by from 49 years (natural attenuation alone) to 22 years (intensive pumping). Conversely, if the mass is greater, a pump-and-treat system will have less of an effect.

Other, potentially less significant sources of uncertainty are:

- *The assumption that concentrations under a pumping scenario will be smaller than concentrations observed under natural flow conditions.* While there is uncertainty in the actual amount, it has been demonstrated in lab studies and the field that increasing the flowrate through a DNAPL source zone will result in lowered concentrations (for example, see Pankow and Cherry, 1996). Therefore we expect some concentration reduction with a pump-and-treat scenario.

- *The assumption that the flow throughout each interval is uniform and that the concentration in each interval can be calculated by averaging each sample point.* These assumptions were used in the mass removal calculation. While there may be some uncertainty in these assumptions, the large number of vertical samples reduces the potential error.

- *The assumption of a first-order decay relationship for the source dissolution rate.* This assumption is based on observations about source decay, and is now used in two EPA peer-reviewed models, BIOSCREEN and BIOCHLOR. While the exact source concentrations curve may not be exactly first order, it will almost certainly fit a first-order decay curve better than assuming constant source concentrations until the source is exhausted. (Note that the use of the first order decay model for the source does not mean that literature-based first-order decay constants for dissolved constituents were used. A source decay constant is based on removal rate and initial source mass, while a biodegradation rate is based on how fast concentrations decay after they leave the source. This study used a source decay approach, and did not use biodegradation rates to estimate source lifetime).

- *No availability effects related to desorption of constituents at low concentrations have been considered.* Slow desorption of non-available fraction of constituents sorbed to aquifer materials will likely reduce the efficiency of any flushing technology. More pronounced effects may be observed for intensive pumping scenarios.

Additional Analysis

A similar analysis was performed for Sites G/H/L using the same calculation approach as was used for Site I (Appendix B). Two cases were performed, and show little impact from a five-year intensive pumping program:

		Estimated Time to Cleanup (years)	% Reduction from Natural Attenuation Only
Case 6	Natural Attenuation Only – Sites G/H/L	434	-
Case 7	5 Yrs of Intensive Pump-and-Treat + Natural Attenuation	427	2% reduction

An evaluation of other constituents present in Sites G/H/I/L groundwater, such as herbicides, pesticides, dioxins, and metals indicates that some constituents will like achieve cleanup goals faster than the VOCs + SVOCs analyzed for this source report, and others may take longer. Ratios of the maximum observed concentrations at Area 1 vs. the Illinois Class I standard for representative constituents provide a general indication of how quickly various constituents may achieve cleanup goals:

CONSTITUENT (Constituent Class)	MAXIMUM CONCENTRATION IN GROUNDWATER (ug/L)	ILLINOIS CLASS I STANDARD (ug/L)	RATIO OF MAX. CONC. / ILLINOIS CLASS I STD.
Chlorobenzene (VOC)	34,000	100	340
2,4-dichlorobenzene (SVOC)	14,000	75	187
Alpha-BHC (Herbicide)	72	0.03	2400
2,4-D (Pesticide)	190	70	2.7
Total PCBs (PCB)	12	0.5	24
Cu (Metal)	3000	650	4.6
Ni (Metal)	7800	100	78
Pb (Metal)	3600	7.5	480
Zn (Metal)	33,000	5000	6.6

On the basis of this general evaluation, alpha-BHC may take longer to achieve cleanup goals than the VOCs+SVOCs, while 2,4-D may take less time. Other factors, such as the mass of each constituent in the source zone and the constituent-specific fate and transport process will determine the ultimate time required to remediate the Area 1 source zones.

KEY POINT: ESTIMATED SOURCE LIFETIMES FOR SEVEN CASES

A planning level source lifetime calculation was done to estimate the relative performance of various remediation schemes. This analysis, while not providing high-confidence estimates of the absolute time to cleanup, does indicate that with an assumed mass of 410,000 kg of VOCs + SVOCs in the saturated zone below Site I, intensive pumping over a 1 to 30 year period does not appear to have an appreciable effect on overall source lifetime (i.e., ≤ 10% reduction). Similar limitations are expected at Sites G/H/L as well.

CONCLUSIONS

Based on the overall groundwater source evaluation at Site I of Sauget Area 1, DNAPL dissolution appears to be the dominant source mechanism. Planning level source lifetime calculations indicate that intensive groundwater pumping will not have an appreciable effect on the overall source lifetime at Site I or at Site G/H/L.

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GROUNDWATER SERVICES, INC.

TABLE 1
SUMMARY OF SITE I GROUNDWATER CONCENTRATIONS BY DEPTH AND COMPARISON TO CONSTITUENT SOLUBILITY
Sampling Period: November to December 1999

Solutia Inc.
Area 1, Sauget and Cahokia, Illinois

Detected Constituent	0-30 ft Depth Conc. (mg/L)	30-70 ft Depth Conc. (mg/L)	70+ ft Depth Conc. (mg/L)	Solubility (mg/L)
VOCs				
1,1-Dichloroethane	0.96	<0.5	<1.0	5,060
1,1-Dichloroethene	0.032 J	<0.5	<1.0	2,250
Benzene	**0.62**	0.19	0.14 J	1,750
Chlorobenzene	8.7	20.0	**34.0**	***<u>472</u>***
cis/trans-1,2-Dichloroethene	**1.2**	0.31	0.001 J	3,500 (7)
Ethylbenzene	**0.87**	0.27	0.074	169
Tetrachloroethene	<0.5	<0.5	0.001 J	200
Toluene	0.018 J	**0.086 J**	0.00089 J	526
Vinyl Chloride	**0.97**	0.32	0.0012 J	2,760
Xylenes, total	<0.5	0.023 J	0.014	186
SVOCs				
1,2,4-Trichlorobenzene	<0.01	<0.5	**2.7**	300
1,2-Dichlorobenzene	0.13	0.32 J	**0.5**	156
1,3-Dichlorobenzene	0.11	**0.29 J**	0.150 J	NA
1,4-Dichlorobenzene	4.4	**10 D**	9.7 D	***<u>73.8</u>***
2,4,5-Trichlorophenol	<0.01	<0.5	0.0018 J	1,200
2,4-Dichlorophenol	<0.01	0.042	0.047 J	4,500
2-Chlorophenol	0.0055 J	0.039	**0.052**	22,000
2-Methylnaphthalene	<0.01	<0.5	0.0013 J	NA
2-Methylphenol (o-cresol)	<0.01	0.003 J	<0.4	26,000
4-Chloroaniline	**4.1 D**	1.7 D	0.018	5,300
Acenaphthene	<0.01	<0.5	0.00033 J	4.24
Carbazole	0.0014 J	0.013	**0.013**	7.48
Di-n-butylphthalate	<0.01	0.00034 J	0.00051 J	11.2
Dibenzofuran	<0.01	0.019 J	<0.4	NA
Diethylphthalate	<0.01	0.0051 J	<0.4	1,080
Fluoranthene	<0.01	0.022 J	<0.4	***<u>0.206</u>***
Hexachlorobenzene	<0.01	<0.5	0.001 J	6.2
N-Nitrosodiphenylamine	0.0053	**0.028**	0.02	35.1
Naphthalene	0.0042 J	0.024	**0.066**	31
Phenanthrene	<0.01	**0.089 J**	0.0013 J	NA
Phenol	<0.01	0.0044 J	<0.4	82,800
bis(2-chloroethyl)ether	0.0011 J	<0.5	<0.4	17,200
bis(2-ethylhexyl)phthalate	0.00069 J	<0.09	<0.072	0.34
Total Detected Conc. (mg/L)	**22.1**	**33.8**	**47.5**	

Notes:

1) Table includes only those compounds detected in at least one groundwater sample for each constituent class. Comparison to solubility includes groundwater sampled at any depth in source area monitoring well.
2) Groundwater samples included are from nearest source area monitoring well only (i.e., AA-I-S1).
3) J = Estimated value. D = Diluted sample. NA = Not available.
4) **Bold** type denotes maximum groundwater concentration by depth.
5) ***<u>Underlined bold italics</u>*** type denotes maximum groundwater concentration exceeds 1% of constit. solubility.
6) Lowest solubility of cis/trans-1,2-Dichloroethene pair indicated.
7) Solubility data from Illinois Tiered Approach to Corrective Action Objectives (TACO).
8) For comparison purposes, non-detectable concentrations are taken as the detection limit shown.

GROUNDWATER SERVICES, INC.

TABLE 2
SUMMARY OF SITE I GROUNDWATER CONCENTRATIONS BY DEPTH VERSUS TCLP WASTE DATA
Sampling Period: November to December 1999

Solutia Inc.
Area 1, Sauget and Cahokia, Illinois

Detected Constituent	0-30 ft Depth Conc. (mg/L)	30-70 ft Depth Conc. (mg/L)	70+ ft Depth Conc. (mg/L)	Max. TCLP Conc. (mg/L)	Media With Max. Conc.
VOCs					
1,1-Dichloroethane	0.96	<0.5	<1.0	NA	-
1,1-Dichloroethene	**<u>0.032 J</u>**	<0.5	<1.0	<0.02	GW
Benzene	**<u>0.62</u>**	0.19	0.14 J	0.14	GW
Chlorobenzene	8.7	20.0	**<u>34.0</u>**	8.9	GW
cis/trans-1,2-Dichloroethene	1.2	0.31	0.001 J	NA	-
Ethylbenzene	0.87	0.27	0.074	NA	-
Tetrachloroethene	<0.5	<0.5	0.001 J	0.29	-
Toluene	0.018 J	0.086 J	0.00089 J	NA	-
Vinyl Chloride	**<u>0.97</u>**	0.32	0.0012 J	<0.04	GW
Xylenes, total	<0.5	0.023 J	0.014	NA	-
SVOCs					
1,2,4-Trichlorobenzene	<0.01	<0.5	2.7	NA	-
1,2-Dichlorobenzene	0.13	0.32 J	0.5	NA	-
1,3-Dichlorobenzene	0.11	0.29 J	0.150 J	NA	-
1,4-Dichlorobenzene	4.4	**<u>10 D</u>**	9.7 D	1.3	GW
2,4,5-Trichlorophenol	<0.01	<0.5	0.0018 J	**<u>1.4</u>**	TCLP
2,4-Dichlorophenol	<0.01	0.042	0.047 J	NA	-
2-Chlorophenol	0.0055 J	0.039	0.052	NA	-
2-Methylnaphthalene	<0.01	<0.5	0.0013 J	NA	-
2-Methylphenol (o-cresol)	<0.01	0.003 J	<0.4	0.014 J	-
4-Chloroaniline	4.1 D	1.7 D	0.018	NA	-
Acenaphthene	<0.01	<0.5	0.00033 J	NA	-
Carbazole	0.0014 J	0.013	0.013	NA	-
Di-n-butylphthalate	<0.01	0.00034 J	0.00051 J	NA	-
Dibenzofuran	<0.01	0.019 J	<0.4	NA	-
Diethylphthalate	<0.01	0.0051 J	<0.4	NA	-
Fluoranthene	<0.01	0.022 J	<0.4	NA	-
Hexachlorobenzene	<0.01	<0.5	0.001 J	<0.05	-
N-Nitrosodiphenylamine	0.0053	0.028	0.02	NA	-
Naphthalene	0.0042 J	0.024	0.066	NA	-
Phenanthrene	<0.01	0.089 J	0.0013 J	NA	-
Phenol	<0.01	0.0044 J	<0.4	NA	-
bis(2-chloroethyl)ether	0.0011 J	<0.5	<0.4	NA	-
bis(2-ethylhexyl)phthalate	0.00069 J	<0.09	<0.072	NA	-

GW Conc. Greater	**5**
TCLP Conc. Greater	**1**

Notes:
1) Table includes only those compounds detected in at least one groundwater sample for each constituent class. Comparison to TCLP waste concentration includes groundwater sampled at any depth in source area monitoring well.
2) Groundwater samples included are collected from nearest source area monitoring well only (i.e., AA-I-S1).
3) J = Estimated value. D = Diluted sample.
4) **Underlined bold** type denotes maximum groundwater concentration or TCLP concentration.
5) TCLP waste data from unsaturated waste samples.
6) NA = Not analyzed.
7) For comparison purposes, non-detectable concentrations are taken as the detection limit shown.

APPENDIX A
DESIGN PUMPING RATE OF HYDRAULIC CONTAINMENT WELLS
Groundwater Alternative D, Intensive Pumping, Sites G, H, I, and L
Sauget Area 1, Sauget and Cahokia, Illinois

PROBLEM: What is estimated pumping rate and number of wells for intensive pumping system for Site I + Sites G/H/L plume?

ASSUMPTIONS:

K = 0.1 cm/sec for middle, deep horizon

b = 80 ft (40 ft middle horizon, 40 ft deep horizon)

i = 0.001 ft/ft

Available drawdown (s) = 15 ft (thickness of shallow unit) (this equals thickness of shallow saturated horizon)

METHOD: Use three different methods to develop a basis for flowrates for an intensive pumping system for the combined Site I + Site G/H/L plume areas.

Method 1. First, an empirical well pumping rate relationship (Driscoll, 1986) based on transmissivity, expected drawdown, and assumptions for other variables in the nonequilibrium (Jacob) equation was used. For this site, a transmissivity of 170,000 gpd/ft was calculated (based on an assumed hydraulic conductivity of 0.1 cm/sec and a saturated thickness of 80 ft for the combined middle and deep horizons).

For unconfined units:

$$\frac{Q}{s} = \frac{T}{1500} \qquad \text{(Eqn. 3, Q in gpm, s in ft, T in gpd/ft)}$$

$$Q(gpm) = \frac{\left(T \ \frac{gpd}{ft}\right)(s \ \ ft)}{1500}$$

$$T = (K)(b_T)$$

$$T = \left(0.1 \frac{cm}{sec}\right)\left(\frac{86400 \text{ sec}}{day}\right)\left(\frac{in}{2.54 \ cm}\right)\left(\frac{ft}{12 \ in}\right)(80 \, ft)\left(\frac{7.48 \ gal}{ft^3}\right)$$

$$T = 170{,}000 \ gpd \, / \, ft$$

$$Q(gpm) = \frac{\left(170{,}000 \ \frac{gpd}{ft}\right)(15 \, ft)}{1500}$$

$Q = 1700\ gpm$

so that:

Q = 1700 gpm for pumping from a regional pumping well

Method 2. Schicht (1965) reports that "It is a general practice of industries and municipalities to place a well in operation and pump it at high rates, often about 1000 gpm." Therefore based on this method:

Q = 1000 gpm for pumping from a regional pumping well

Method 3. Schicht also reports the specific capacity from three wells in T2N, R10W (where Area 1 is located) as being 152.5, 188, and 158 gpm/ft, respectively. By averaging these specific capacities (166 gpm/ft), and multiplying by an assumed drawdown of 15 ft, a pumping rate of **2490 gpm** is obtained. As would be expected, use of specific capacities results in a wide range of predicted well pumping rates due to the effects of well construction, well condition, and local hydrogeologic conditions.

Q = 2500 gpm for pumping from a regional pumping well

RESULT: These calculation approaches suggest that a regional pumping well could yield from 1000 to 2500 gpm in the Area 1 location.

Therefore, it was assumed that the total pumping rate of any intensive pumping system would also be in this range, although the flow would be distributed among several wells. Therefore the following conceptual design was developed:

Qtotal = 1500 gpm total flowrate (based on lower-middle range of flowrate estimates to be conservative)

APPENDIX B
RELATIVE SOURCE LIFETIME OF AREA I UNDER NATURAL ATTENUATION VS. INTENSIVE PUMP AND TREAT
Groundwater Alternative D, Intensive Pumping, Sites G, H, I, and L

Sauget Area 1, Sauget and Cahokia, Illinois

I. SOURCE LIFETIME CALCULATION: SITE I

PROBLEM: What is relative source lifetime of Site I under natural attenuation vs. intensive pump and treat conditions?

ASSUMPTIONS:

1. Source Volume = $(1400\,ft)(500\,ft)(95\,ft)\left(7.48\frac{gal}{ft^3}\right)(0.35)(0.05)(0.01) = 87{,}000\ gals$

   Source Mass = $(87{,}000\,gal)\left(\frac{3.78L}{gal}\right)\left(\frac{1.25kg}{L}\right) = 410{,}000\ kg$

   Where: width = 1400 ft; length = 500 ft; sat. thickness = 95 ft; porosity = 0.35; assumed residual saturation = 0.05; fraction of source containing residual saturation = 0.01. (see text)

2. Current Mass Removal Rate: 7000 kg/yr (Natural Attenuation) (see text)
   Initial Intensive Pumping Removal Rate: 17,500 kg/yr (Intensive Pump-and-Treat) (see text).

3. Case 1 Natural Attenuation Only – Site I
   Case 2 1 Yr of Intensive Pump-and-Treat + Natural Attenuation – Site I
   Case 3 5 Yrs of Intensive Pump-and-Treat + Natural Attenuation – Site I
   Case 4 10 Yrs of Intensive Pump-and-Treat + Natural Attenuation – Site I
   Case 5 30 Yrs of Intensive Pump-and-Treat + Natural Attenuation – Site I

4. Starting concentration under natural conditions: 20 mg/L (representative of middle and deep units in 1999). Starting concentration under pumping conditions: 5.5 mg/L (due to mass-transfer effects for deep and middle units; factor of 3.6 reduction).

5. Assumed ending concentration: 0.005 mg/L (MCL for several constituents).

MODEL:

$$\frac{C_{(t)}}{C_{(now)}} = e^{-k_s t}$$ (from BIOSCREEN and BIOCHLOR models; see text)

$$k_s = \frac{Mass\ Removal\ Rate\ (kg/yr)}{Mass\ (kg)} \quad (k_s = \text{source decay constant})$$

$$t(yr) = \frac{-\ln\left(\frac{C_t}{C_{now}}\right)}{k_s}$$

Model Applied to Case 1: Natural Attenuation Only

$$k_s = \frac{7000\frac{kg}{yr}}{410{,}000\ kg} = 0.017\ yr^{-1}$$

$$t(yr) = \frac{-\ln\left(\frac{0.005}{20}\right)}{0.017}$$

RESULT (Case 1): ***t = 488 years***

Model Applied to Case 2: Intensive Pump and Treat With 1 Year of Pumping

$$k_s = \frac{17{,}500\frac{kg}{yr}}{410{,}000\ kg} = 0.043\, yr^{-1}$$

$$\frac{C_{1yr}}{C_{now}} = e^{-k_s t}$$

$$\frac{C_{1yr}}{5.5\ mg/L} = e^{-(0.043 yr^{-1})(1yr)}$$

$$C_{1\ yr} = 5.27\ mg/L$$

Time to cleanup after 1 yr of pumping is finished, with 3.6-times increase in concentration due to rebound (5.27 mg/L * 3.6 = 19.0 mg/L).

$$\frac{0.005\ mg/L}{19.0\ mg/L} = e^{-(0.017 yr^{-1})(t\ yrs)}$$

$$t = \frac{-\ln\left(\frac{0.005}{19.0}\right)}{0.017}$$

$$t = 485\ years$$

RESULT (Case 2): Total time to cleanup (Intensive Pump-and-Treat for 1 yr) = 1 + 485 = **486 years**

Model Applied to Case 3: Intensive Pump and Treat With 5 Years of Pumping

$$k_S = \frac{17{,}500 \frac{kg}{yr}}{410{,}000\ kg} = 0.043\ yr^{-1}$$

$$\frac{C_{5\ yrs}}{C_{now}} = e^{-k_s\, t}$$

$$\frac{C_{5\ \mathrm{yrs}}}{5.5\ \mathrm{mg/L}} = e^{-(0.043\ \mathrm{yr}^{-1})(5\ \mathrm{yrs})}$$

$$C_{5\ yrs} = 4.44\ mg/L$$

Time to cleanup after 5 yrs of pumping is finished, with 3.6-times increase in concentration due to rebound (4.44 mg/L * 3.6 = 16.0 mg/L).

$$\frac{0.005\ mg/L}{16.0\ mg/L} = e^{-\left(0.017\ yr^{-1}\right)(t\ yrs)}$$

$$t = \frac{-\ln\left(\frac{0.005}{16.0}\right)}{0.017}$$

$$t = 475\ years$$

RESULT (Case 3): Total time to cleanup (Intensive Pump-and-Treat for 5 yrs) = 5 + 475 = **480 years**

Model Applied to Case 4: Intensive Pump and Treat With 10 Years of Pumping

$$k_s = \frac{17,500 \frac{kg}{yr}}{410,000\ kg} = 0.043\,yr^{-1}$$

$$\frac{C_{10yrs}}{C_{now}} = e^{-k_s\,t}$$

$$\frac{C10yrs}{5.5\ mg/L} = e^{-(0.043yr^{-1})(10yrs)}$$

$$C10\ _{yrs} = 3.58\ mg/L$$

Time to cleanup after 10 yrs of pumping is finished, with 3.6-times increase in concentration due to rebound (3.58 mg/L * 3.6 = 12.9 mg/L).

$$\frac{0.005\ mg/L}{12.9\ mg/L} = e^{-\left(0.017yr^{-1}\right)(t\ yrs)}$$

$$t = \frac{-\ln\left(\frac{0.005}{12.9}\right)}{0.017}$$

$$t = 462\ years$$

RESULT (Case 4): Total time to cleanup (Intensive Pump-and-Treat for 10 yrs) = 10 + 462 = **472 years**

Model Applied to Case 5: Intensive Pump and Treat With 30 Years of Pumping

$$k_s = \frac{17,500 \frac{kg}{yr}}{410,000\ kg} = 0.043\,yr^{-1}$$

$$\frac{C_{30yrs}}{C_{now}} = e^{-k_s\,t}$$

$$\frac{C30yrs}{5.5\ mg/L} = e^{-(0.043yr^{-1})(30yrs)}$$

$$C30\ _{yrs} = 1.51\ mg/L$$

Time to cleanup after 30 yrs of pumping is finished, with 3.6-times increase in concentration due to rebound (1.51 mg/L * 3.6 = 5.4 mg/L).

$$\frac{0.005\ mg/L}{5.4\ mg/L} = e^{-\left(0.017yr^{-1}\right)(t\ yrs)}$$

$$t = \frac{-\ln\left(\frac{0.005}{5.4}\right)}{0.017}$$

$$t = 411\ years$$

RESULT (Case 5): Total time to cleanup (Intensive Pump-and-Treat for 30 yrs) = 30 + 411 = **441 years**

CONCLUSION: Comparison of Cleanup Times – Site I

Case 1	Natural Attenuation Only	488 years
Case 2	1 Yr of Intensive Pump-and-Treat + Natural Attenuation	486 years
Case 3	5 Yrs of Intensive Pump-and-Treat + Natural Attenuation	480 years
Case 4	10 Yrs of Intensive Pump-and-Treat + Natural Attenuation	472 years
Case 5	30 Yrs of Intensive Pump-and-Treat + Natural Attenuation	441 years

II. SOURCE LIFETIME CALCULATION: SITES G/H/L

PROBLEM: What is relative source lifetime of Sites G/H/L under natural attenuation vs. intensive pump and trea conditions?

ASSUMPTIONS:

1. Assume Sites G/H/L together have approximately same dimensions as Site I, with 1% of the starting mass as Site I, because VOC+SVOC concentrations are much lower leaving Sites G/H/L (~ 0.20 mg/L) than Site I (~ 20 mg/L).

2. Assume ratio of source mass at Sites G/H/L and Site I are proportional to ratio of representative concentrations and width parallel to groundwater flow leaving Sites G/H/L and Site I.

$$\text{Source Mass} = \left(\frac{\text{representative conc. Sites G/H/L mg/L}}{\text{representative conc. Site I mg/L}}\right)(\text{Est. Mass Site I kg})$$

$$= \left(\frac{0.20 \text{ mg/L}}{20 \text{ mg/L}}\right)(410{,}000 \text{ kg})$$

$$= 4100 \text{ kgs}$$

3. Current Mass Removal Rate: Assume 1% of Site I mass removal rate based on ratio of representative concentration at Site I (20 mg/L VOC+SVOC) to representative concentration at Sites G/H/L (0.20 mg/L VOC+SVOC).

Natural Source Removal Rate =

$$\left(\frac{\text{representative conc. Sites G/H/L mg/L}}{\text{representative conc. Site I mg/L}}\right)\left(\frac{\text{Width Sites G/H/L}}{\text{Width Site I}}\right)(\text{Est. Mass Removal Rate Site I kg/yr})$$

$$= \left(\frac{0.20 \text{ mg/L}}{20 \text{ mg/L}}\right)\left(\frac{750 \text{ ft}}{1400 \text{ ft}}\right)(7000 \text{ kg/yr})$$

$$= 35 \text{ kg/yr}$$

Use Initial Intensive Pumping Removal Rate of 2.5 times 34 kg/yr (Intensive Pump-and-Treat= 87.5 kg/yr

4. Run analysis for two cases:

   Case 6 Natural Attenuation Only – Site G/H/L

   Case 7 5 Yrs of Intensive Pump-and-Treat + Natural Attenuation – Site G/H/L

5. Starting concentration under natural conditions: 0.20 mg/L (representative of middle and deep units in 1999). Starting concentration under pumping conditions: 0.056 mg/L (due to mass-transfer effects for middle and deep units; factor of 3.6 reduction).

6. Assumed ending concentration: 0.005 mg/L (MCL for several constituents).

MODEL:

$$\frac{C_{(t)}}{C_{(now)}} = e^{-k_s t}$$ (from BIOSCREEN and BIOCHLOR models; see text)

$$k_s = \frac{Mass\ Removal\ Rate\ (kg/yr)}{Mass\ (kg)}$$ (k_s = source decay constant)

$$t(yr) = \frac{-\ln\left(\frac{C_t}{C_{now}}\right)}{k_s}$$

Model Applied to Case 6: Natural Attenuation Only

$$k_s = \frac{35\frac{kg}{yr}}{4100\ kg} = 0.0085\ yr^{-1}$$

$$t(yr) = \frac{-\ln\left(\frac{0.005}{0.2}\right)}{0.0085}$$

RESULT (Case 6): ***t = 434 years***

Model Applied to Case 7: Intensive Pump and Treat With 5 Years of Pumping

$$k_s = \frac{87.5\frac{kg}{yr}}{4100\ kg} = 0.021\,yr^{-1}$$

$$\frac{C_{1yr}}{C_{now}} = e^{-k_s\,t}$$

Starting concentration under pumping conditions: 0.056 mg/L (due to mass-transfer effects for deep and middle units; factor of 3.6 reduction).

$$\frac{C_{5yr}}{0.056\ mg/L} = e^{-(0.021yr^{-1})(5yrs)}$$

$$C_{5\ yr} = 0.050\ mg/L$$

Time to cleanup after 5 yrs of pumping is finished, with 3.6-times increase in concentration due to rebound (0.050 mg/L * 3.6 = 0.18 mg/L).

$$\frac{0.005\ mg/L}{0.18\ mg/L} = e^{-\left(0.00085 yr^{-1}\right)(t\ yrs)}$$

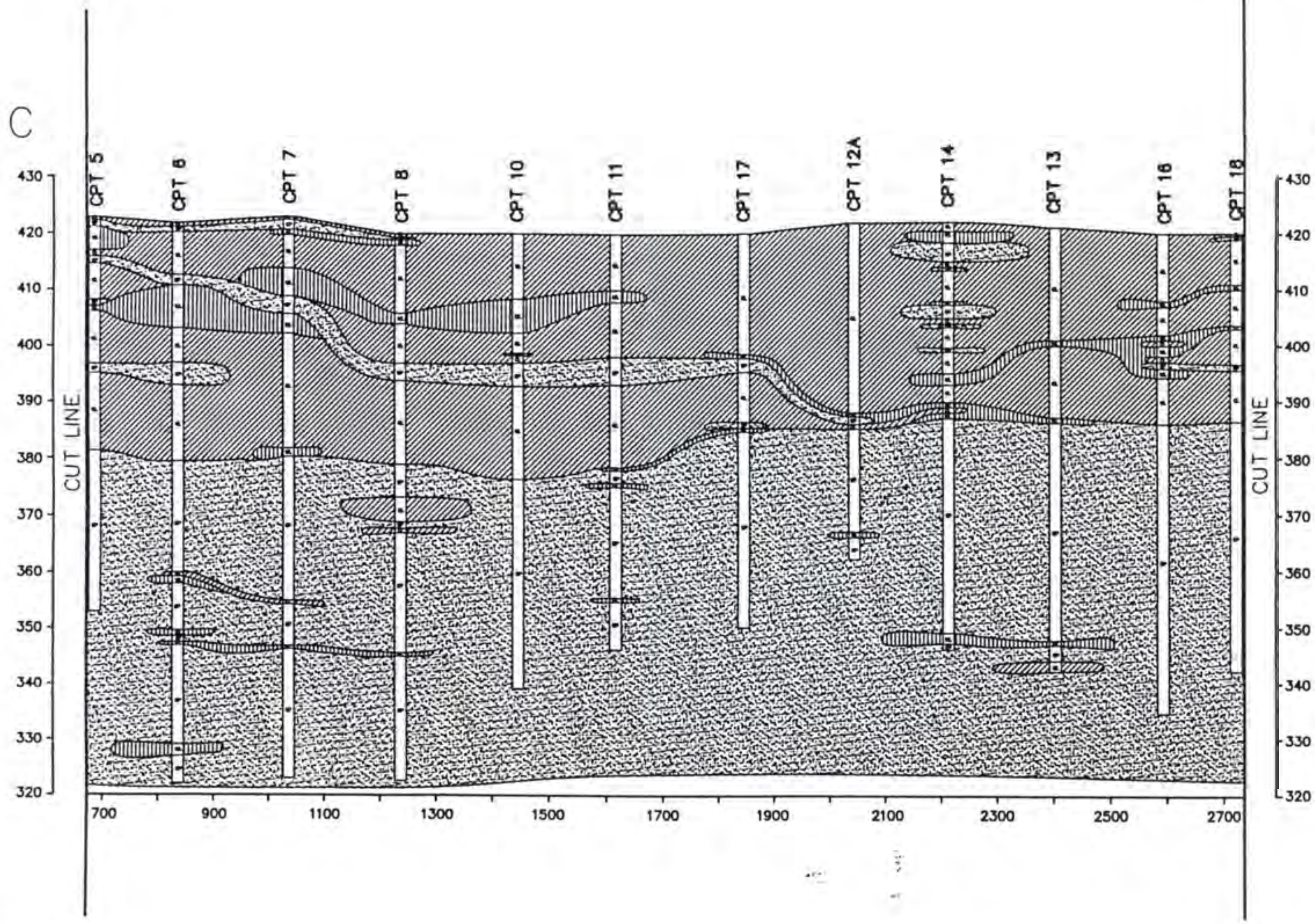
$$t = \frac{-\ln\left(\frac{0.005}{0.18}\right)}{0.0085}$$

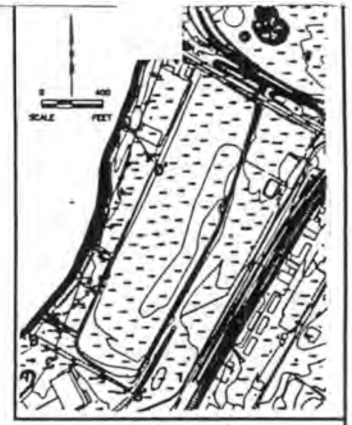
$$t = 422\ years$$

RESULT (Case 7): Total time to cleanup (Intensive Pump-and-Treat for 5 yrs) = 5 + 422 = **427 years**

CONCLUSION: Comparison of Cleanup Times – Sites G/H/L

Case 6	Natural Attenuation Only – Sites G/H/L	434 years
Case 7	30 Yrs of Intensive Pump-and-Treat + Natural Attenuation – Sites G/H/L	427 years





KEY MAP

LEGEND

▨	CLAY
▥	SILT
■	SAND
CPT-1	CONE PENETROMETER TEST NUMBER

VERTICAL SCALE 1"=10'
HORIZONTAL SCALE 1"=100'

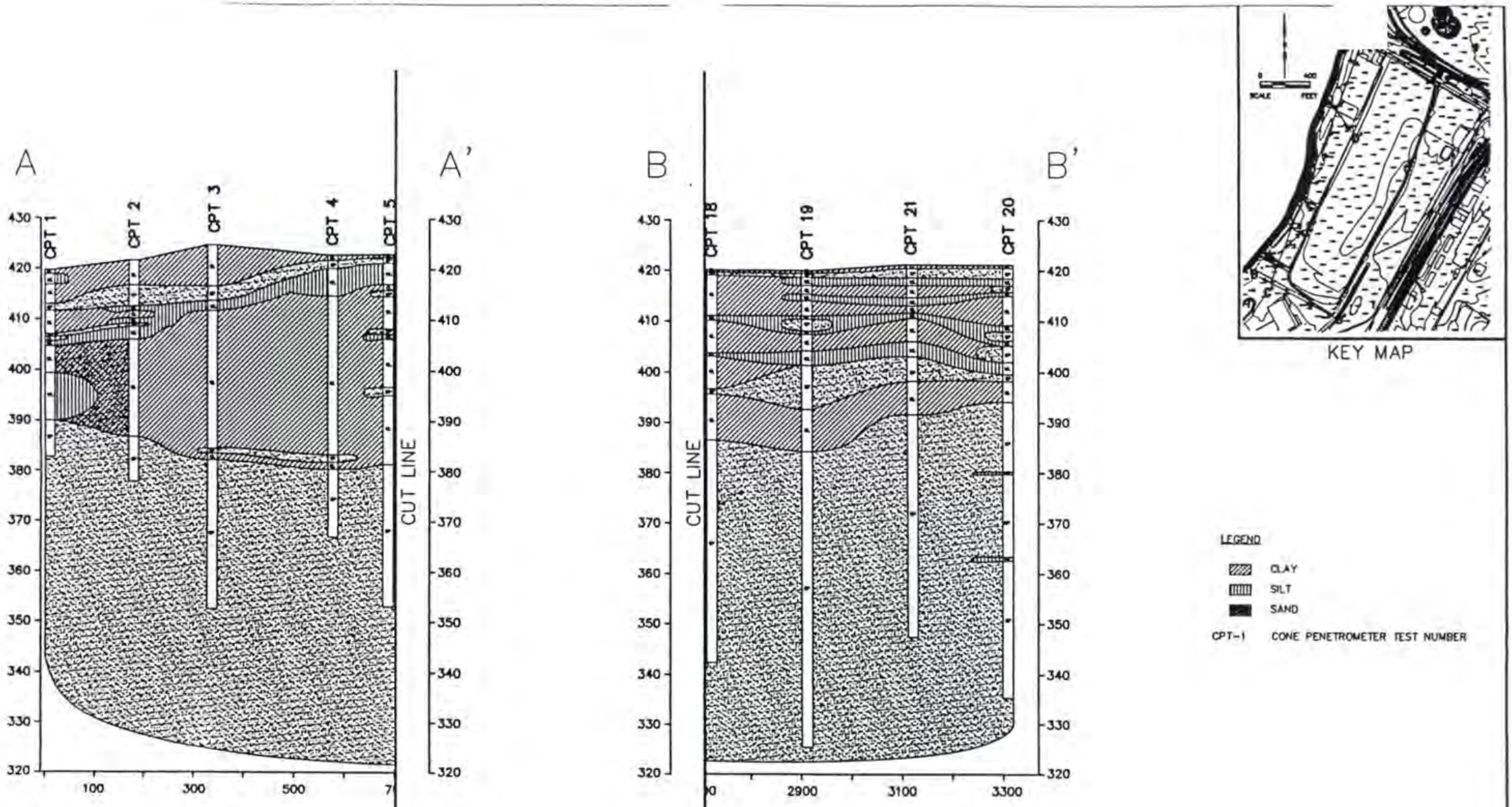
KRUMMRICH GROUNDWATER MIGRATION CONTROL PREDESIGN INVESTIGATION
SAUGET, ILLINOIS

Geological Crossection

Date:	Project Number:	Drawing Number:
9/12/01	2320010058.00	6
Drawn by: DJD	Design by: MRF/SJS	Checked by: MRF

URS

Attachment A.2.1
Geologic Cross-Section
URS

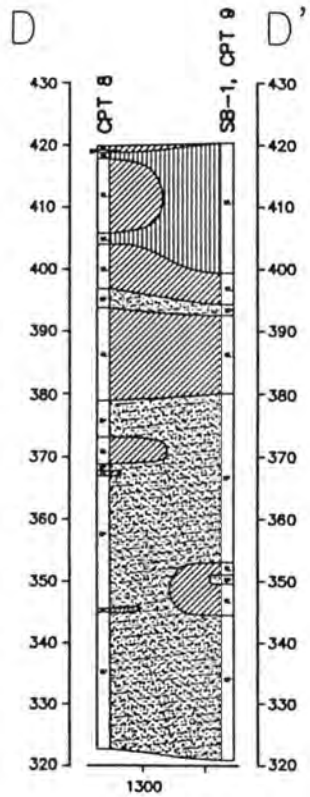


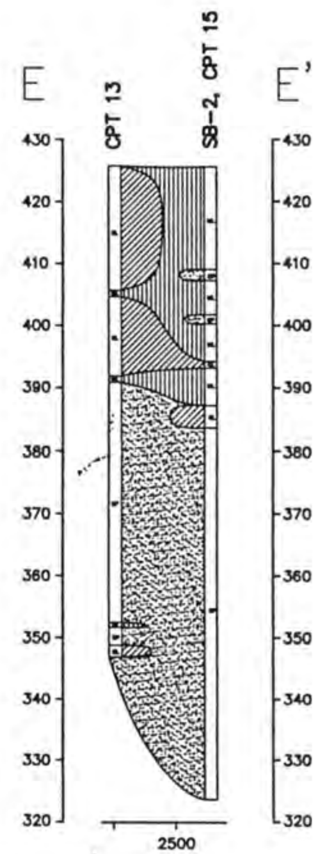
VERTICAL SCALE 1"=10'
HORIZONTAL SCALE 1"=100'

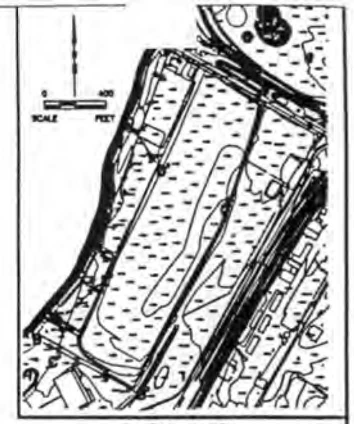
KRUMMRICH GROUNDWATER MIGRATION CONTROL PREDESIGN INVESTIGATION SAUGET, ILLINOIS		
Geological Crossection		
Date: 9/12/01	Project Number: 23200010058.00	Drawing Number: 5
Drawn by: DJD	Design by: MRF/SJS	Checked by: MRF

URS

Attachment A.2.2
Geologic Cross-Section
URS







KEY MAP

LEGEND

CLAY
SILT
SAND
CPT-1 CONE PENETROMETER TEST NUMBER

VERTICAL SCALE 1"=10'
HORIZONTAL SCALE 1"=100'

KRUMMRICH GROUNDWATER MIGRATION CONTROL PREDESIGN INVESTIGATION
SAUGET, ILLINOIS

Geological Crossection

Date: 9/12/01	Project Number: 2320010058.00	Drawing Number: 7
Drawn by: DJD	Design by: MRF/SJS	Checked by: MRF

URS

Attachment A.2.3
Geologic Cross-Section
URS

CONTENTS

FIGURES

TABLES (Continued)

4. Estimates of Aquifer Characteristics Obtained Through Interpretation of Observation Well Drawdown Data From the June 1992 Aquifer Test, Sauget Site R, Monsanto Company, Sauget, Illinois.

5. Estimate of Average Daily Ground-Water Discharge to Mississippi River, Sauget Site R, Monsanto Company, Sauget, Illinois.

APPENDIX G

GROUNDWATER FLOW CONDITIONS

Groundwater flow at Site R has been monitored routinely since 1983. Additional information on groundwater flow and aquifer characteristics of the three hydrogeologic zones within the unconsolidated aquifer was developed during RI activities in 1992. These activities included collecting water-level measurements under static conditions and conducting an aquifer test. This information was used to supplement previous data and to calibrate a three-dimensional groundwater flow model (Appendix H). Section 1 discusses groundwater flow conditions; Section 2 provides results of the aquifer test; and Section 3 provides a discussion of groundwater discharge calculations.

1.0 GROUNDWATER FLOW

Section 1.1 provides a description of groundwater flow conditions based on data collected prior to December 1992. Section 1.2 discusses results of modeling performed to assess the impact of the 1993 Mississippi River flood on the groundwater system.

1.1 NORMAL RIVER STAGES

As discussed in Section 2.6 of the RI Report (Historical Groundwater Use and Flow Patterns), regional groundwater flow in the three hydrogeologic zones is to the west, towards the Mississippi River. Water levels measured on June 3, 1992 in the shallow, intermediate, and deep zones are shown on Figures 1, 2, and 3, respectively. These data are summarized in Table 1.

Figure 1 shows that a groundwater mound exists in the shallow zone at Site R. The existence of this mound has been previously documented in the RI work plan. It is apparently due to low permeability units beneath the area that reduce drainage rates from the shallow zone after periods of precipitation or high river stage. Groundwater flows to the east and south from the mound, but must eventually flow west toward the river. Historical data and the groundwater model (Appendix H) indicate that the eastern flow reaches a stagnation point (where the eastward flow meets the regional westward flow) which is generally between Site R and the levee. Its exact location depends on the magnitude of the regional westward flow and river stage. At the stagnation point, water from the shallow zone flows downward into the intermediate zone. Water which flows south from the mound eventually turns to the west under the influence of the regional flow patterns.

Both the easterly and southerly flow from the mound are included in the model. The easterly flow is included in the intermediate zone estimate of groundwater discharge to the river. Wells screened in the intermediate zone adjacent to the river encounter this flow. Shallow wells along the river in the southern portion of Site R and in the Expanded Study Area encounter the southern flow.

Figures 2 and 3 show that groundwater flow in the intermediate and deep zones on June 3, 1992 was toward the river. Water-level data from well clusters screened in the intermediate and deep zones (GM-27B and GM-27C, P-8 and GM-56C, and GM-28B and GM-28C) indicates that there is an upward gradient from the deep zone to the intermediate zone (Table 1). This is to be expected because these wells are adjacent to the Mississippi River, which is a major groundwater discharge boundary. Groundwater flows from the lower portion of the aquifer up toward the river.

During periods of high river stage, when the river rises higher than the water table, gradients in the intermediate and deep zones are reversed. Flow in all three zones is toward the east, but eventually reaches a stagnation point where the eastward gradient equals the westward regional gradient. This "riverbank storage effect" can last from several days to a few weeks. The response of all three zones to varying river stages was demonstrated in hydrographs provided in the RI Work Plan (Geraghty & Miller 1990).

Analytical data from the well cluster located adjacent to the flood control levee (GM-62A, GM-62B, and GM-62C) indicate that there has been little, if any, transport of constituents from Site R to the east. The concentrations of total VOCs and total SVOCs are less than 150 ug/L in each of these wells. These concentrations are several orders of magnitude lower than the concentrations detected in Site R wells.

1.2 FLOOD CONDITIONS

In order to assess the impact of extreme conditions, such as those in the flood of 1993, a scenario which simulated even worse conditions was run on the model. A flood stage of 48 ft was assumed to last for 60 days. The flow field at the end of the 60-day period was then used to estimate the flow velocities to the east. The actual flood crest was 49.5 ft on August 1, 1993, and river levels dropped by 10 ft (to 39.5 ft) within two weeks.

The modeling results estimate that under the extreme conditions simulated, groundwater in the intermediate zone would travel approximately 6.5 ft/day. In the deep zone groundwater would travel approximately 8.3 ft/day. Water levels in the shallow zone did not reach equilibrium in the 60-day period modeled. Water-level measurements obtained from wells east of the flood wall on July 24, 1993 (when the river stage was 46.5 ft) were used to calculate a groundwater velocity of 0.06 ft/day in the shallow zone.

Within the actual groundwater flow environment, constituents dissolved in the groundwater would move more slowly than the predicted groundwater velocities because various factors such as adsorption and biodegradation can retard their movement. No retardation coefficients were considered in the modeled scenario.

2.0 AQUIFER TEST

An aquifer test was conducted to provide site-specific hydraulic characteristics necessary to calibrate the three-dimensional groundwater flow model for the area and to calculate concentrations of constituents discharging to the Mississippi River for use in the risk assessment. During June 15 through 19, 1992, a step-drawdown test, constant-rate aquifer test, and recovery test were conducted. The site-specific aquifer coefficients determined from this testing include transmissivity, hydraulic conductivity, and storage coefficient.

2.1 FIELD TESTING

Prior to testing, two 6,000-gallon Calgon carbon adsorption units were delivered to the site, set up in series, and filled with 40,000 pounds of activated carbon to treat the discharge water on site. Piping was then installed from the well to the carbon units and from the carbon unit discharge line to a line which was connected to the American Bottoms treatment facility. The American Bottoms facility issued a permit for this discharge before testing was initiated.

A step-drawdown test was conducted to evaluate the optimum pumping rate for the constant-rate aquifer test. Based on this test, a flow rate of 350 gallons per minute (gpm) was selected for the constant-rate test. A network of 22 wells was monitored on a regular basis using three different types of monitoring equipment. Pressure transducers were used to monitor water-level changes in 16 monitoring wells, automatic Steven's water-level recorders were used on three wells, and manual measurements were collected in three wells. Table 2 provides a summary of the method used to monitor each well. The water-level measurements collected during the test are provided in Attachment A.

During the test, water samples were collected from the carbon unit influent, lead vessel effluent, and final effluent after 6, 24, and 48 hours, for laboratory analysis of volatile organic compounds (VOCs), semivolatile organic compounds (SVOCs), and herbicides. The final effluent sample collected 24 hours into the test was also analyzed for cyanide, ammonia, metals, and pesticides. Analysis was performed by Savannah Laboratories, Savannah, Georgia. Field analyses of the phenol in lead vessel effluent were conducted to monitor for breakthrough.

At approximately 12 hours into the test, the river stage began to rise as a result of a storm event that had occurred upriver several days earlier (Figure 4). Water levels within the wells began to rise in response to the river, and the cone of depression that had been established began to diminish.

Drawdown in the intermediate and deep zones was plotted after 550 minutes of pumping to show the effect of pumping prior to the impact of rising river stage on these zones (Figure 5). Review of Figure 5 shows that approximately 1 ft of drawdown was induced at a distance of 100 to 150 ft from pumping well TW-1, and drawdown appeared to extend to the site boundaries. Approximately 0.2 ft of drawdown was observed in wells along the eastern border of Site R, approximately 0.4 ft of drawdown was observed in wells along the northern boundary, and approximately 0.1 ft of drawdown was observed in wells along the southern boundary of the site.

After approximately 1000 minutes of pumping, the rising river stage reduced drawdown in intermediate and deep zone wells, and the cone of influence decreased in size (Figure 6). Along the southern boundary of the area of influence, water levels rose to 0.2 ft above the static level in well GM-55C and 0.59 ft in well GM-28C. Along the northern boundary, water levels rose to 1.2 ft above the static level. The effect of the rising river stage is less apparent in the intermediate and deep zone wells in the vicinity of well TW-1, where drawdown data did not change significantly (Figure 6). Eastern perimeter wells exhibited increased drawdowns at 1000 minutes and were apparently unaffected by elevated river stage. This is most likely due to their distance from the river.

After 51 hours of pumping, the constant-rate drawdown test was completed, and recovery measurements were collected for 4 hours. This information was used to confirm the results of the drawdown test. Recovery water-level measurements are provided in Attachment A.

2.2 METHODS OF EVALUATION

Different types of aquifers respond to pumping in different ways. Several analytical solutions were used to evaluate the test data, to determine whether the aquifers could be characterized as confined or semi-confined.

Unconfined aquifer conditions were analyzed using the non-equilibrium method of Neuman or the methods of Theis and Cooper-Jacob with Jacob's correction for reduction in saturated thickness. The applicability of the semi-confined (leaky) solution of aquifer conditions was analyzed using the non-equilibrium method of Hantush, with storage in the overlying unpumped aquifer zone.

Except for the Jacob distance-drawdown solution, all of the methods were applied with the support of AQTESOLV, a Geraghty & Miller Modeling Group aquifer test analysis software package. Data utilized by the Jacob distance-drawdown solution were plotted on semilog paper.

AQTESOLV is an interactive, menu driven program that provides graphical curve matching techniques for quick and efficient analysis of aquifer test data. The option was utilized in which the analyst interactively matches type curves to the time-drawdown data directly on the computer screen. Data relevant to the configuration of the aquifer test are presented in Table 3.

2.2.1 Theis Method

If an unconfined aquifer does not exhibit a delayed water-table response, then the Theis Method for unsteady flow in confined aquifers can be applied once the drawdown data are corrected as follows:

$$S' = S - \frac{S^2}{2m}$$

where S' = equivalent confined aquifer drawdown

S = observed drawdown under unconfined conditions

m = aquifer thickness (pretesting)

Correction of drawdown data was unnecessary, however, because dewatering of the aquifer was insignificantly small in relation to the total saturated thickness of the aquifer. The Theis type-curve exhibited a close match with the log-log plots of drawdown versus time for the intermediate zone wells.

2.2.2 Neuman Method

Water levels near a pumping well in unconfined aquifers often tend to decline at a slower rate than that described by the Theis solution. Log-log plots of time-drawdown exhibit a three phase S-shape curve due to the phenomenon of "delayed water-table response." The second phase is characterized by gravity drainage of the pore spaces that is not instantaneous. A delay in the release of this stored water causes the increase of drawdown to slow with time, and thus deviate from the Theis curve (Kruseman and de Ridder 1990). Data from the three deep zone wells corresponded to the flow regime described by the Neuman solution. The applicability of the Neuman method to this aquifer is based on the premise that slow drainage from low permeability zones and horizontal-to-vertical anisotropy cause a delayed yield of water released from storage.

2.2.3 Hantush Method

When a well is pumped in a leaky aquifer, the well discharge comes from storage within the aquifer, vertical leakage from stored water in the aquitard, and leakage through the aquitard from the overlying unpumped aquifer. The leaky-confined aquifer analytical solution was also applied to the time-drawn data from the deep zone wells because aquifer deposits in the water-table zone beneath Site R consist of poorly sorted, fine grained material of low permeability, and drawdown in this zone was negligible.

2.2.4 Jacob (Distance-Drawdown) Method

Simultaneous drawdown measurements in several observation wells, each at a different distance from the pumped well, were plotted on semilog paper to show the straight line distance-drawdown relationship. This distance-drawdown graph was used to calculate the aquifer transmissivity and storativity. Distance-drawdown graphs were plotted for data from groups of intermediate and deep zone wells to determine the aquifer characteristics for those zones.

2.3 INTERPRETATION OF RESULTS

2.3.1 Water-Table Zone

Observation wells screened in the poorly sorted, fine grained material of this zone recorded maximum drawdown ranging from 0.08 to 0.15 ft. This small drawdown was not sufficient to establish drawdown behavior from natural fluctuations in water-level elevations. Thus, the aquifer characteristics of the water-table zone were not estimated through the analysis of aquifer test data.

2.3.2 Intermediate Zone

Time-drawdown measurements in the intermediate zone wells exhibited the characteristic shape of the Theis type-curve (Figures 7 through 11). The decline in measured drawdown beginning about 800 minutes after the start of pumping indicates the recovery in water-levels induced by aquifer recharge from the river. Table 4 presents trasmissivity values for all observation wells that produced a sufficient drawdown response; values ranged from about 22,000 to 38,200 square feet per day (ft^2/day). Values of storativity calculated with the Theis method range from .004 to .013 (Table 4). Estimates of transmissivity obtained with the Theis solution were closely reproduced with the Cooper-Jacob (semilog) method for Well B-24C (Figure 12) and the Neuman method for Well B-26B (Figure 13).

The Jacob distance-drawdown method was applied to a group of intermediate zone wells (P-5, P-9, P-10, B-26B) at 10, 100, and 500 minutes into the aquifer test. This method yielded inconsistent results that ranged from 24,702 ft^2/day to 51,463 ft^2/day (Figure 14). These estimates were not considered to be as reliable as individual well analysis.

2.3.3 Deep Zone

Several minutes into the aquifer test, the increase in drawdown in the deep zone wells began to slow and deviate from the Theis type-curve. This phenomenon, in which the drawdown curve approaches horizontal, is characteristic of the aquifer response to delayed gravity drainage of water released from storage (Figure 15 and 16). Calculation of transmissivities with the Neuman solution for deep zone Wells GM-56C and GM-57C were 23,961 ft^2/day and 29,736 ft^2/day. Under normal circumstances, the time-drawdown curve increases in slope and once again conforms to the Theis curve. However, the water-level recovery in the observation wells induced by recharge from the river masked the typical third phase of the Neuman curve.

The effect of leakage through an overlying confining unit on drawdown is comparable to that of delayed drainage. Therefore, the Hantush (leaky confined) solution was used to determine if it was the appropriate analytical model for this aquifer. The deep zone transmissivities calculated with the Hantush method yielded one low estimate of 15,580 ft^2/day (Well GM-56C), and two more representative estimates of 30,859 ft^2/day for Well GM-57C, and 31,162 ft^2/day for Well GM-28C (Figures 17 through 19). However, the Hantush type-curve did not fit the time-drawdown data as well as the Neuman type-curve. Although the Hantush solution yielded similar results to the Neuman solution, its applicability to this aquifer system for analysis of the aquifer test is not the appropriate selection. The Neuman theoretical model identifies most closely with this aquifer system and provides the best interpretation of the time-drawdown data.

The Jacob distance-drawdown analysis was also performed on the group of deep zone wells (GM-28C, GM-56C, and GM-57C) at 10, 100, and 500 minutes into the aquifer test (Figure 20). The method yielded consistent results but the transmissivities were lower than estimates computed for individual time-drawdown plots. Values of transmissivity with the Jacob method ranged from 17,154 ft^2/day to 22,055 ft^2/day. The distance-drawdown results for the intermediate and deep zone wells were lower than individual well estimates. However, the groundwater flow model was calibrated with transmissivity values based on the higher estimates obtained from individual well plots. Thus, simulated remedial pumping rates will produce conservative estimates of capture zones since they are based on values in the higher range of transmissivity estimates.

3.0 GROUNDWATER DISCHARGE CALCULATIONS

As one of the first steps of the risk assessment, a list of chemicals of concern (COC) was selected for the groundwater at Site R. In order to complete the evaluation of risks associated with exposure to river water affected by the ground water, predicted concentrations of the COCs in the river were calculated. Geraghty & Miller used the groundwater model described in Appendix H and the concentrations of the COCs in the wells to complete these calculations.

Several steps were involved in the process. First, because the rate of groundwater discharge to the river changes with varying river stage, data were obtained from the U.S. Army Corps of Engineers (COE) which show the daily percent frequency of occurrence for every river stage on record in 1-ft increments, i.e., the percent of days in a given period that each river stage occurred. The data included the 130-year period from January 1861 to December 1991.

Using these data, a range of river stages was selected for the discharge calculations. The lower limit of this range was 374 ft above mean sea level (msl), the lowest river stage on record. The upper limit of the range was 410 ft above msl. Groundwater level data and the model indicate that the hydraulic gradient in the aquifer reverses above this level, so there would be no discharge to the river. These river stages and their frequency of occurrence are shown in columns 1 and 2 of Table 5.

The model was used to predict the groundwater discharge to the river at each river stage in the range. A separate calculation was done for each of the three hydrogeologic zones (Columns

3, 5, and 7 of Table 5). These predicted discharge rates at each river stage were then multiplied by the frequency of occurrence for that stage. These products (columns 4, 6, and 8 of Table 5) were summed to obtain a weighted average daily discharge for each aquifer zone. This represents the average volume of ground water which flows into the river each day from each aquifer zone along the entire length of the landfill (2,000 ft). In the next step, the length of the river frontage was divided into segments. Each hydrogeologic zone was treated separately and was divided into one segment for each well screened in that zone. The percent of river frontage represented by each well segment was multiplied by the average daily discharge for that aquifer zone and then by the concentration in that well of each COC. These products were summed to obtain a weighted average daily loading of each COC to the river for each aquifer zone. These were then summed across the three zones to obtain a total average daily loading to the river for each COC.

To obtain the predicted concentration of each COC in the river, these daily loadings will be divided by the flow rate in the river. Both average exposure and reasonable maximum exposure (RME) scenarios will be considered in the risk assessment. Calculations of the river concentrations of each COC will be shown in the risk assessment.

The discharge across all zones for all river stages was summed in Column 9 of Table 5. This number (795,000 gallons/day) will be used for calculating percent dilution in the evaluation of aquatic hazard indices in the risk assessment).

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Table 1. Water-Level Elevations in Monitoring Wells at Sauget Site R, June 3, 1992, Monsanto Company, Sauget, Illinois.

Well No.	Measuring Point Elevation (1)	Depth to Water (2)	Water Level Elevation (1)
Shallow Wells			
B-21A	428.53	29.93	398.60
B-22A	428.16	28.79	399.37
B-24A	422.49	22.17	400.32
B-25A	428.47	30.95	397.52
B-26A	423.71	26.37	397.34
B-28A	423.04	25.25	397.79
B-29A	429.03	31.97	397.06
B-31B	421.68	28.45	393.23
P-6	421.78	24.92	396.86
P-7	421.82	22.96	398.86
P-14	424.36	24.93	399.43
GM-62A	425.75	31.99	393.76
GM-65A	424.11	32.66	391.45
GM-66A	422.73	27.79	394.94
Intermediate Wells			
GM-27B	426.04	36.63	389.41
GM-28B	423.88	34.15	389.73
B-21B	428.37	36.12	392.25
B-22B	428.16	37.55	390.61
B-23B	428.17	33.97	394.20
B-24B	422.28	31.70	390.58
B-24C	422.52	32.02	390.50
B-25B	427.35	34.84	392.51
B-26B	423.62	33.29	390.33
B-27B	425.83	34.26	391.57
B-28B	423.08	32.55	390.53
B-29B	429.06	36.69	392.37
B-30B	430.52	38.02	392.50
B-31C	421.88	28.70	393.18
P-1	423.11	33.18	389.93
P-2	423.15	33.26	389.89
P-3	423.43	33.27	390.16
P-4	421.82	31.70	390.12
P-5	422.12	32.31	389.81
P-8	421.79	32.02	389.77
P-9	423.14	33.38	389.76
P-10	423.43	33.19	390.24
P-11	422.30	32.68	389.62
P-12	423.75	34.45	389.30
P-13	424.32	34.90	389.42
GM-62B	426.16	32.42	393.74
GM-66B	423.20	34.05	389.15
TW-1	423.14	32.47	390.67
Deep Wells			
GM-27C	426.76	36.60	390.16
GM-28C	423.78	33.98	389.80
GM-55C	422.88	32.32	390.56
GM-56C	422.16	31.16	391.00
GM-57C	424.02	34.06	389.96
GM-62C	427.03	33.30	393.73
GM-66C	423.46	34.37	389.09
Bedrock Wells			
GM-106	424.82	29.73	395.09
B-102	423.84	31.74	392.10

(1) Elevation in feet above mean sea level.
(2) Depth to water in feet below measuring point.
The water-level for Well B-21A may be representative of a water level in the 2-ft section of blank casing at the bottom of the well, and not representative of the water table zone. This water level was not used in the groundwater model.

Table 2. Wells Monitored During the June 1992 Aquifer Test, Sauget Site R, Monsanto Company, Sauget, Illinois.

Well Number	Pressure Transducer	Stevens Recorder	Manual Measurement
Water-Table Zone			
P-7	X	-	-
B-24A	X	-	-
B-25A	X	-	-
B-26A	X	-	-
Intermediate Zone			
P-5	X	-	-
P-8	X*	-	-
P-9	X	-	-
P-10	X	-	-
B-24C	X*	-	-
B-25B	X	-	-
B-26B	X*	-	-
B-30B	-	-	X
B-31C	-	-	X
GM-27B	-	X	-
GM-28B	X	-	-
Test Well 1	X	-	-
Deep Zone			
GM-27C	-	X	-
GM-28C	X	-	-
GM-55C	-	-	X
GM-56C	X*	-	-
GM-57C	X	-	-
Bedrock Zone			
GM-106	-	X	-

* Backup transducer was installed

Table 3. Data Used to Define the Configuration of the June 1992 Aquifer Test, Sauget, Illinois.

Well Number		Distance From Pumped Well to Observation Well	Maximum Drawdown	Saturated Thickness	Well Depth	Depth From Static Water Level to Top of Screen	Depth From Static Water Level to Bottom of Screen
	Water-Table Zone						
B-24A		118	.08	90	27.5	0	2.6
B-25A		625	.07	90	35.2	0	1.7
B-26A		355	.15	90	33.2	0	3.8
P-7		102	.05	90	33	2.7	7.7
	Intermediate Zone						
B-24C		118	.98	90	69	22.9	32.9
P-10		143	.97	90	54	11.4	16.4
P-9		104	.99	90	50	7.7	12.7
B-25B		625	.33	90	49.5	0.8	10.8
P-5		272	.36	90	54.5	13.3	18.3
B-26B		433	.46	90	49.8	2.4	12.4
GM-28B		772	.22	90	93	34.4	54.4
P-8		112	–	90	53.5	12.5	17.5
TW-1*		0	34.8	90	108	41.5	73.5
	Deep Zone						
GM-57C		368	.77	90	116	60	80
GM-28C		772	.21	90	107	51	71
GM-56C		150	1.21	90	111	58.1	78.1

* Pumping Rate = 350 gpm; screened interval is from the lower intermediate zone to the upper deep zone.

– Not reported due to problems with pressure transducer.

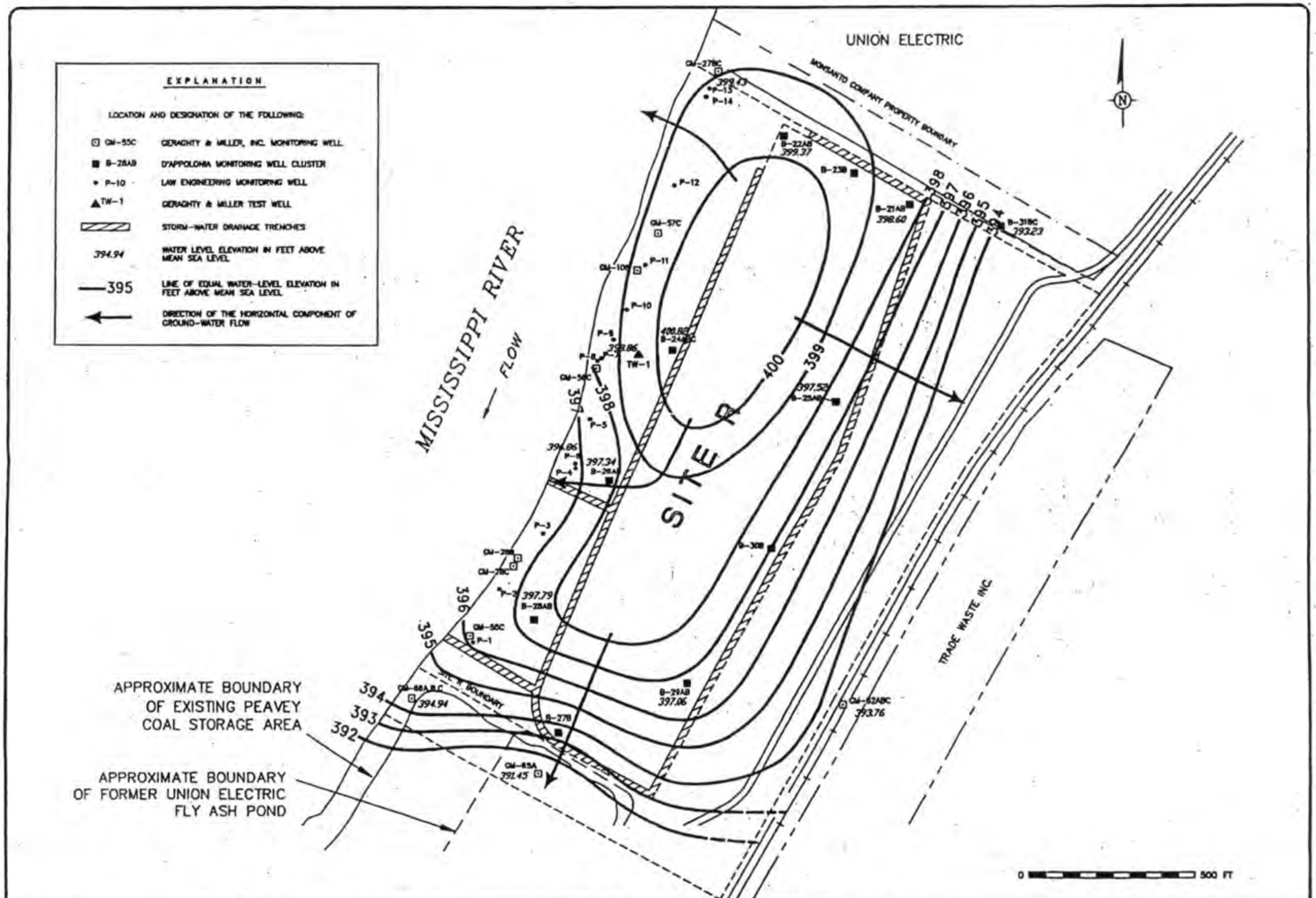
Table 4. Estimates of Aquifer Characteristics Obtained Through Interpretation of Observation Well Drawdown Data From the June 1992 Aquifer Test, Sauget Site R, Monsanto Company, Sauget, Illinois.

Well Number	Method	Aquifer Test	Transmissivity Ft^2/Min	Transmissivity Ft^2/Day	Hydraulic Conductivity Ft/Day	Storage Coefficient
Intermediate Zone						
P-5	Theis	Unconfined	26.55	38,232	425	.012
P-9	Theis	Unconfined	15.28	22,003	244	.0134
P-10	Theis	Unconfined	15.19	21,874	243	.0083
B-24C	Theis	Unconfined	23.8	34,272	381	.0042
	Cooper-Jacob	Unconfined	22.53	32,443	360	.0045
B-26B	Theis	Unconfined	22.22	31,996	356	.0065
	Neuman	Unconfined	20.3	29,232	325	.007 (Sy)
Deep Zone						
GM-57C	Hantush	Leaky	21.43	30,859	343	.0004
	Neuman	Unconfined	20.65	29,736	330	.0055 (Sy)
GM-56C	Hantush	Leaky	10.82	15,580	173	.0013
	Neuman	Unconfined	16.64	23,961	266	.016 (Sy)
GM-28C	Hantush	Leaky	21.64	31,162	346	.0001
Distance-Drawdown Evaluation						
Intermediate Wells	Jacob	Unconfined				
10 minutes			35.74	51,463	572	.0055
100 minutes			23.8	34,309	381	.0025
500 minutes			17.2	24,702	274	.0146
Deep Wells	Jacob	Unconfined				
10 minutes			15.32	22,055	245	.0005
100 minutes			12.61	18,163	202	.0028
500 minutes			11.91	17,154	191	.0104

Table 5. Estimate of Average Daily Ground-Water Discharge to Mississippi River, Sauget Site R, Monsanto Company, Sauget, Illinois.

1 River-Stage Elevation (ft above MSL)	2 Frequency of Occurence (% of days)	3 Q Shallow	4 Q x % Shallow	5 Q Intermediate	6 Q x % Intermediate	7 Q Deep	8 Q x % Deep	9 Total Q x %
374	0.2	0.00024	0.00000048	0.6496	0.0012992	0.6228	0.0012456	0.00254528
375	0.5	0.00037	0.00000185	0.634	0.00317	0.608	0.00304	0.00621185
376	1.5	0.0005	0.0000075	0.6184	0.009276	0.5932	0.008898	0.0181815
377	2.2	0.00063	0.00001386	0.6028	0.0132616	0.5784	0.0127248	0.02600026
378	2.3	0.00076	0.00001748	0.5872	0.0135056	0.5636	0.0129628	0.02648588
379	3	0.00089	0.0000267	0.5716	0.017148	0.5488	0.016464	0.0336387
380	3.8	0.00102	0.00003876	0.556	0.021128	0.534	0.020292	0.04145876
381	3.9	0.00109	0.00004251	0.5406	0.0210834	0.5194	0.0202566	0.04138251
382	4.6	0.00116	0.00005336	0.5252	0.0241592	0.5048	0.0232208	0.04743336
383	4.2	0.00124	0.00005208	0.5098	0.0214116	0.4902	0.0205884	0.04205208
384	4.9	0.00131	0.00006419	0.4944	0.0242256	0.4756	0.0233044	0.04759419
385	4.9	0.00138	0.00006762	0.479	0.023471	0.461	0.022589	0.04612762
386	5.5	0.00142	0.0000781	0.4638	0.025509	0.4462	0.024541	0.0501281
387	4.5	0.00145	0.00006525	0.4486	0.020187	0.4314	0.019413	0.03966525
388	4.4	0.00149	0.00006556	0.4334	0.0190696	0.4166	0.0183304	0.03746556
389	4.1	0.00152	0.00006232	0.4182	0.0171462	0.4018	0.0164738	0.03368232
390	4.1	0.00156	0.00006396	0.403	0.016523	0.387	0.015867	0.03245396
391	3.5	0.00158	0.0000553	0.3878	0.013573	0.3724	0.013034	0.0266623
392	3.5	0.00159	0.00005565	0.3726	0.013041	0.3578	0.012523	0.02561965
393	3.5	0.00161	0.00005635	0.3574	0.012509	0.3432	0.012012	0.02457735
394	3.6	0.00162	0.00005832	0.3422	0.0123192	0.3286	0.0118296	0.02420712
395	2.7	0.00164	0.00004428	0.327	0.008829	0.314	0.008478	0.01735128
396	2.9	0.00164	0.00004756	0.3116	0.0090364	0.2992	0.0086768	0.01776076
397	2.5	0.00164	0.000041	0.2962	0.007405	0.2844	0.00711	0.014556
398	2.2	0.00164	0.00003608	0.2808	0.0061776	0.2696	0.0059312	0.01214488
399	2.1	0.00164	0.00003444	0.2654	0.0055734	0.2548	0.0053508	0.01095864
400	1.8	0.00164	0.00002952	0.25	0.0045	0.24	0.00432	0.00884952
401	1.8	0.00163	0.00002934	0.235	0.00423	0.2256	0.0040608	0.00832014
402	1.3	0.00161	0.00002093	0.22	0.00286	0.2112	0.0027456	0.00562653
403	1.4	0.0016	0.0000224	0.205	0.00287	0.1968	0.0027552	0.0056476
404	1.1	0.00158	0.00001738	0.19	0.00209	0.1824	0.0020064	0.00411378
405	1.2	0.00157	0.00001884	0.175	0.0021	0.168	0.002016	0.00413484
406	0.9	0.00154	0.00001386	0.1602	0.0014418	0.1537	0.0013833	0.00283896
407	0.9	0.00152	0.00001368	0.1454	0.0013086	0.1394	0.0012546	0.00257688
408	1.1	0.00149	0.00001639	0.1306	0.0014366	0.1251	0.0013761	0.00282909
409	0.9	0.00147	0.00001323	0.1158	0.0010422	0.1108	0.0009972	0.00205263
410	0.8	0.00144	0.00001152	0.101	0.000808	0.0965	0.000772	0.00159152
Weighted Average Daily Discharge	98.3		0.00135768 (Shallow)		0.4047248 (Intermediate)		0.3888442 (Deep)	0.79492665 (Total)

) - Rate of ground-water discharge to river (million gallons per day).



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Environmental Services

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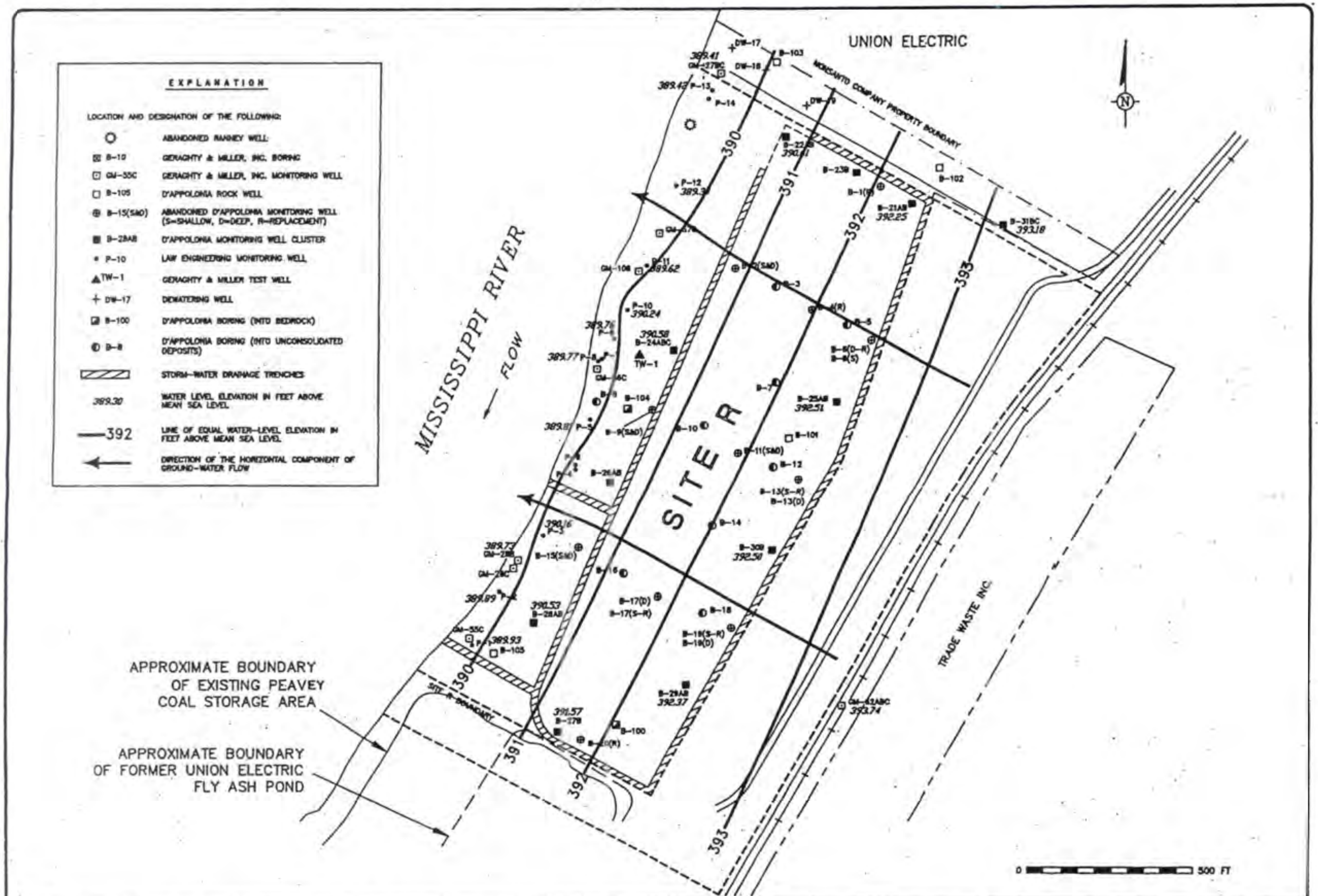
USE TO VERIFY FIGURE REPRODUCTION SCALE

PROJECT NO.: NY0421	FILE NO. 1988
DRAWING: WAT-LEV	PLOT SIZE:
DRAFTED BY: VC/GS	DATE: 8-8-94
CHECKED BY: LM	DATE:
APPROVED BY: DC	DATE:

WATER LEVEL ELEVATIONS IN THE SHALLOW ZONE ON JUNE 3, 1992

MONSANTO CHEMICAL COMPANY
SAUGET SITE R, ILLINOIS

FIGURE 1



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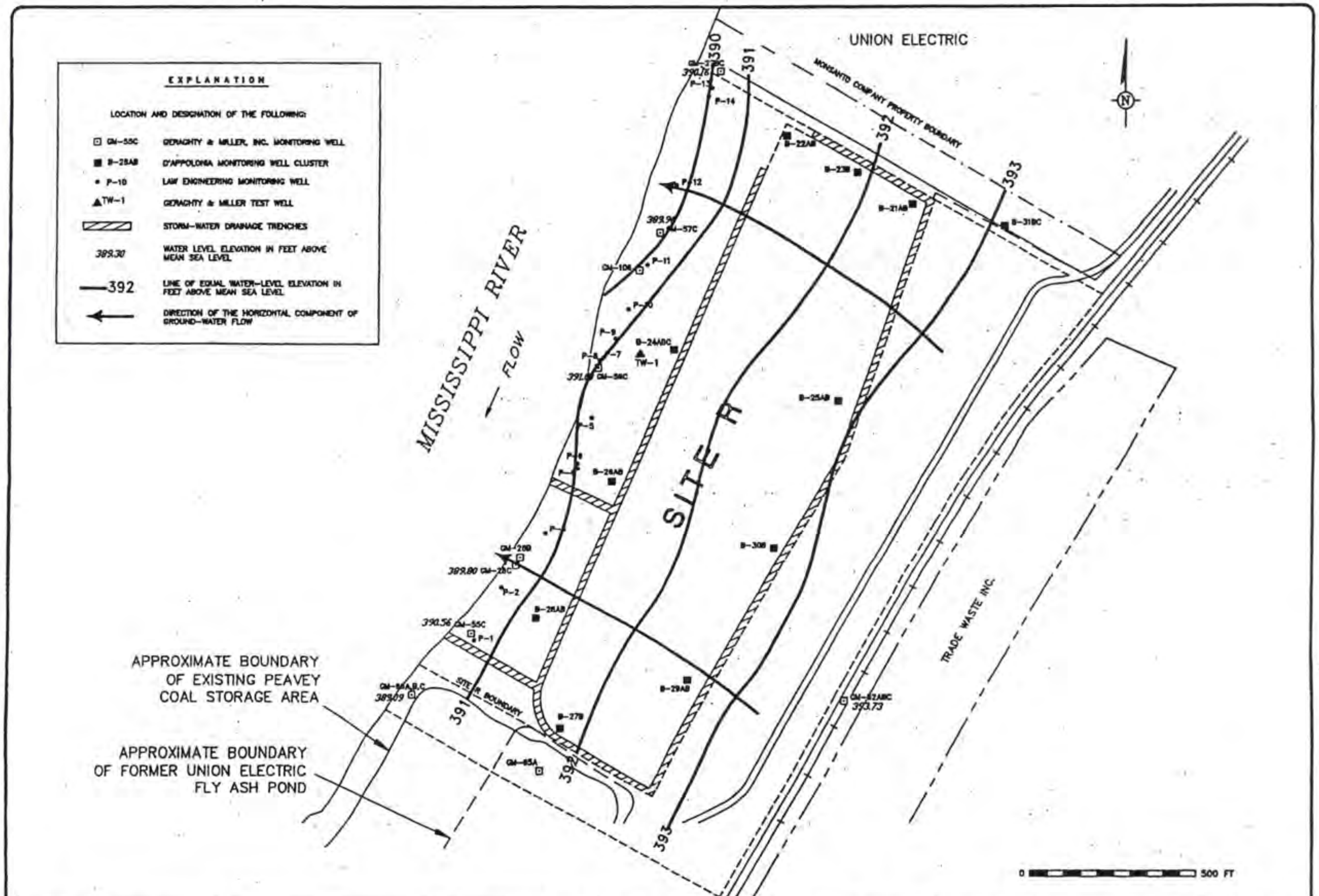
DRAWING	PLOT SIZE
DRAFTED BY	DATE: 10-30-92
CHECKED BY	DATE:
APPROVED BY	DATE:

WATER-LEVEL ELEVATIONS IN THE INTERMEDIATE ZONE ON JUNE 3, 1992

MONSANTO COMPANY
SAUGET, ILLINOIS

FIGURE 2



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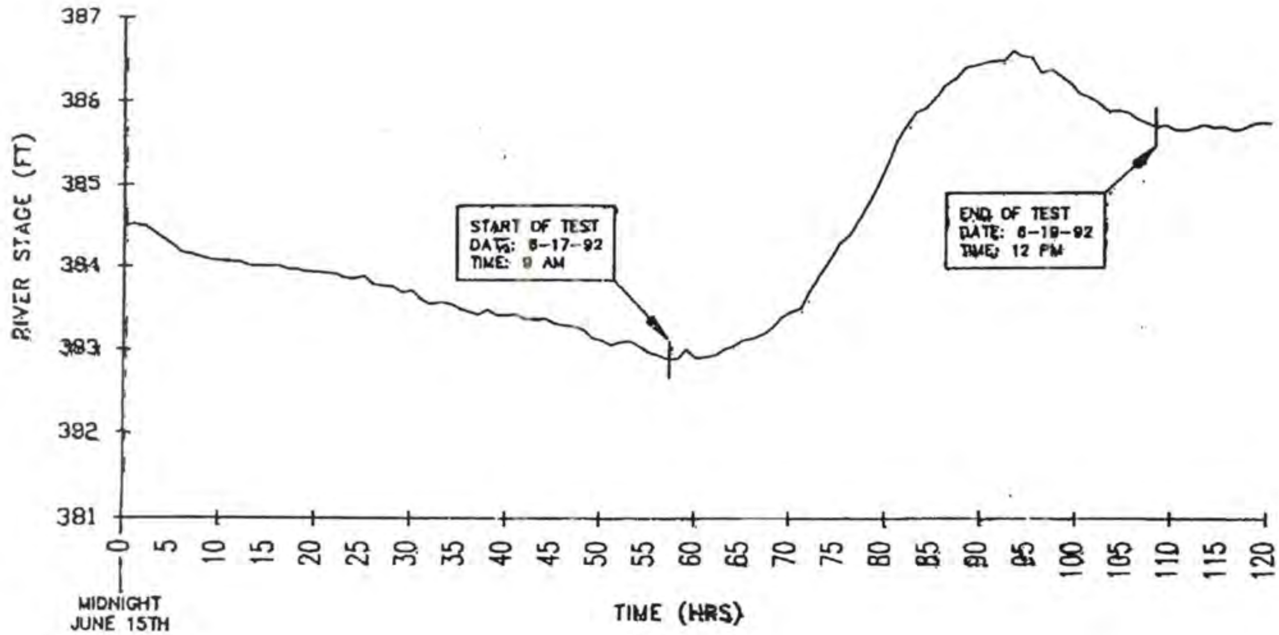
PROJECT NO.:	FILE NO.:
DRAWING:	PLOT SIZE:
DRAFTED BY: VC	DATE:
CHECKED BY: LM	DATE:
APPROVED BY: DC	DATE:

WATER-LEVEL ELEVATIONS IN THE DEEP ZONE ON JUNE 3, 1992

MONSANTO COMPANY
SAUGET, ILLINOIS

FIGURE 3

DWG DATE: 2-9-93 | PROJECT NO.: NY64207 | FILE NO.: 1556 | DRAWING: MISSRIV | CHECKED: S. FELDMAN | APPROVED: T. KRAMER | DRAFTER: V.C./G.S.

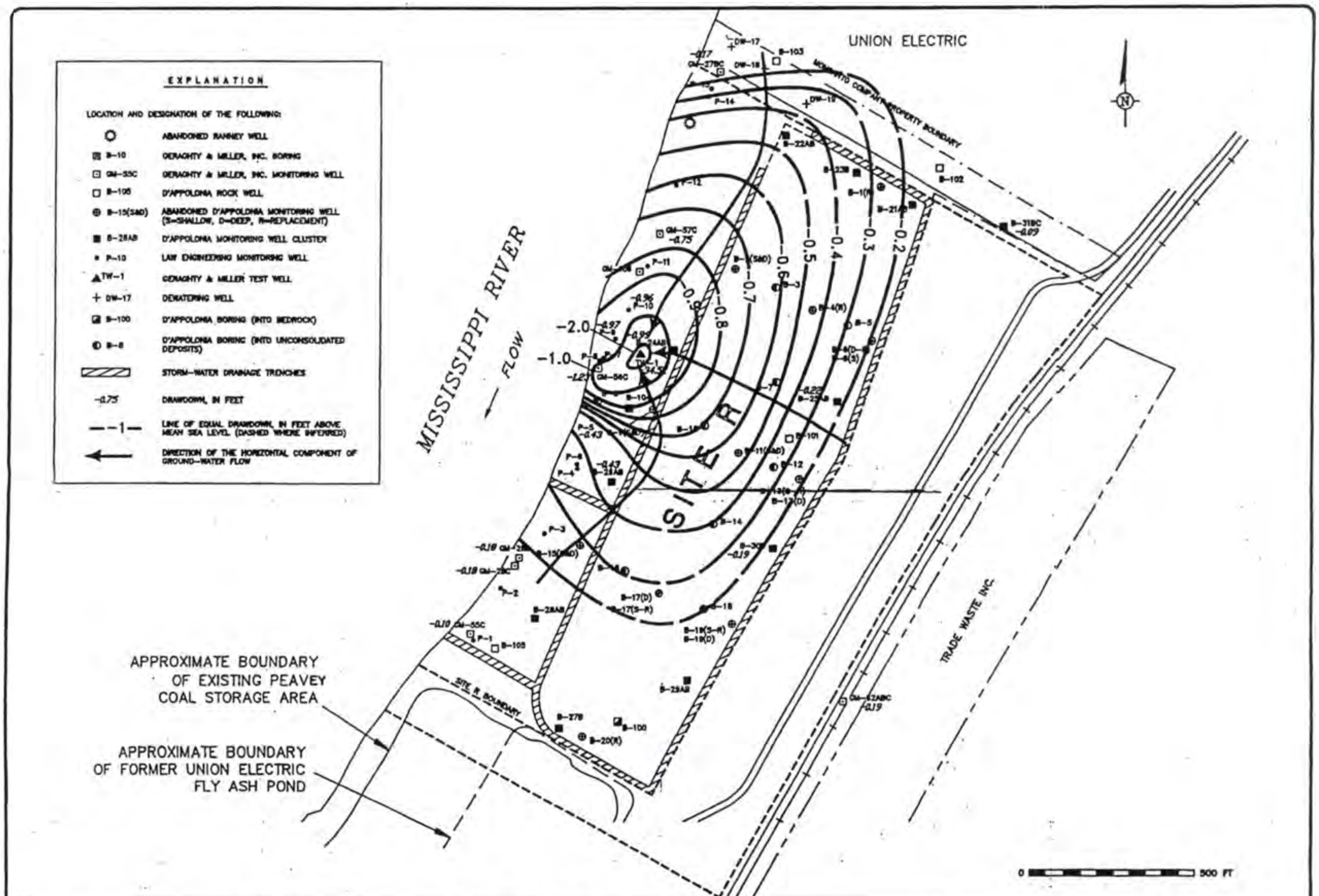


GERAGHTY & MILLER, INC.
Environmental Services

MISSISSIPPI RIVER STAGE AT THE ST. LOUIS STATION FROM JUNE 15 THROUGH 19, 1992

MONSANTO
SAUGET, ILLINOIS

FIGURE
4

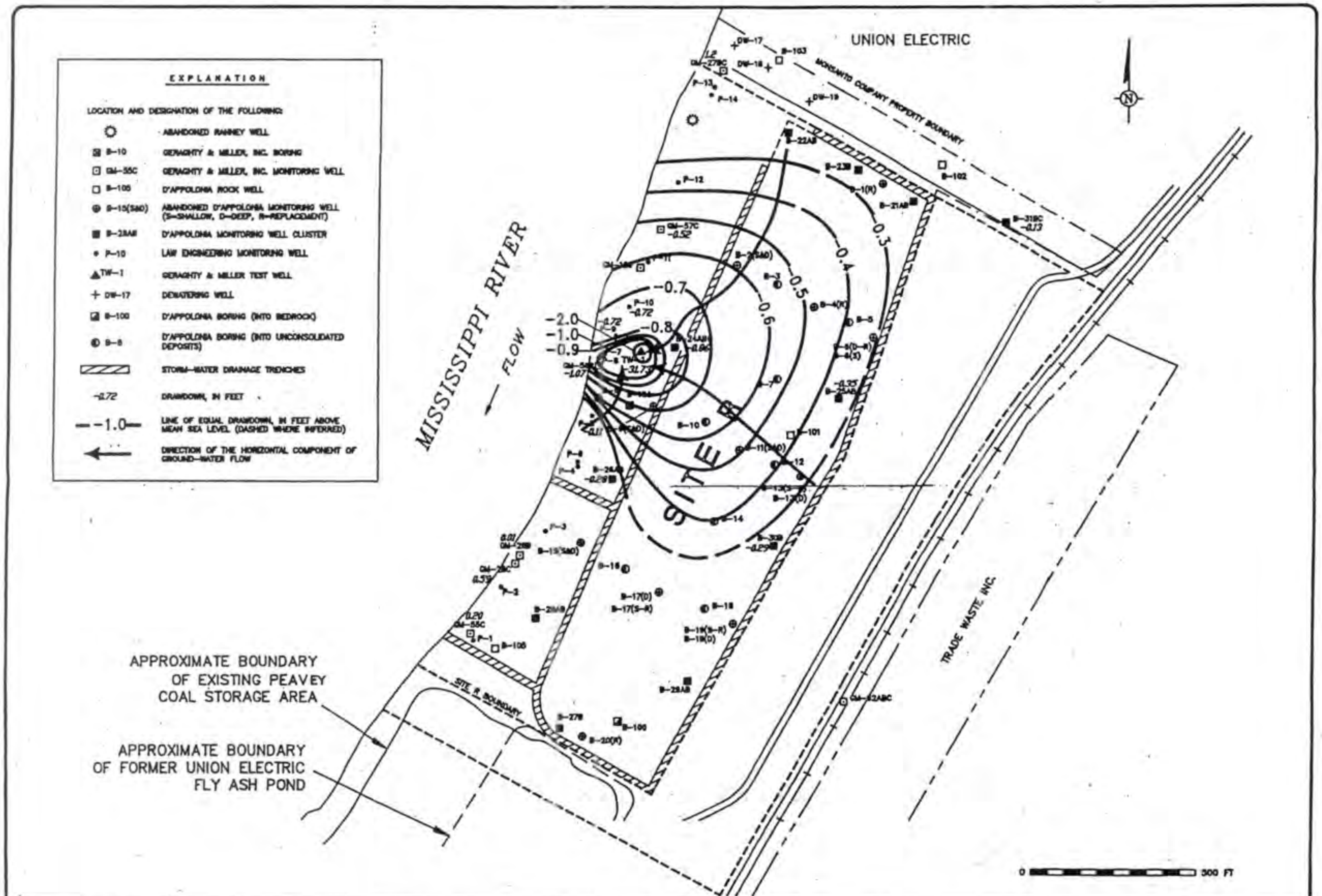


GERAGHTY & MILLER, INC. Environmental Services	DRAWING CONFIDENTIAL: THIS DRAWING AND ALL INFORMATION CONTAINED THEREON IS AND SHALL REMAIN THE PROPERTY OF GERAGHTY & MILLER, INC. AS AN INSTRUMENT OF PROFESSIONAL SERVICE. THIS INFORMATION SHALL NOT BE USED IN WHOLE OR IN PART WITHOUT THE FULL KNOWLEDGE AND PRIOR WRITTEN CONSENT OF GERAGHTY & MILLER, INC.	SCALE VERIFICATION: THIS BAR REPRESENTS ONE INCH ON THE ORIGINAL DRAWING. USE TO VERIFY FIGURE REPRODUCTION SCALE		DRAWDOWN IN THE INTERMEDIATE AND DEEP AQUIFERS AFTER 550 MINUTES OF PUMPING ON JUNE 17, 1992 — MONSANTO COMPANY, SAUGET, ILLINOIS	FIGURE 5

DRAWDOWN IN THE INTERMEDIATE AND DEEP AQUIFERS AFTER 550 MINUTES OF PUMPING ON JUNE 17, 1992

MONSANTO COMPANY
SAUGET, ILLINOIS

FIGURE 5



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CHECKED BY: LM	DATE:
APPROVED BY: DG	DATE:

DRAWDOWN IN THE INTERMEDIATE AND DEEP AQUIFERS AFTER 1,000 MINUTES OF PUMPING ON JUNE 18, 1992

MONSANTO COMPANY
SAUGET, ILLINOIS

FIGURE 6

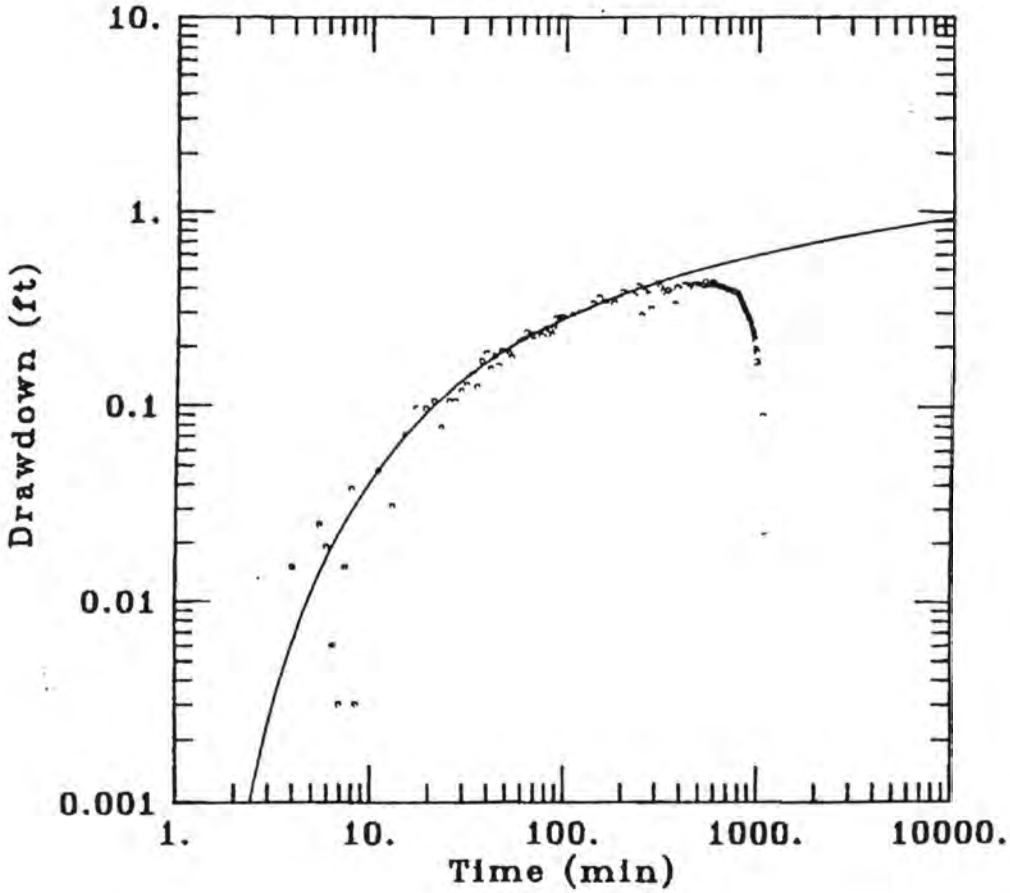
GERAGHTY & MILLER

Client: MONSANTO

Project No.: NY64207

Location: SAUGET, ILLINOIS

P–5



DATA SET:
P-5.AQT
07/08/92

AQUIFER TYPE:
Unconfined
SOLUTION METHOD:
Theis
TEST DATE:
JUNE 17, 1992
TEST WELL:
TW-1
OBS. WELL:
P-5

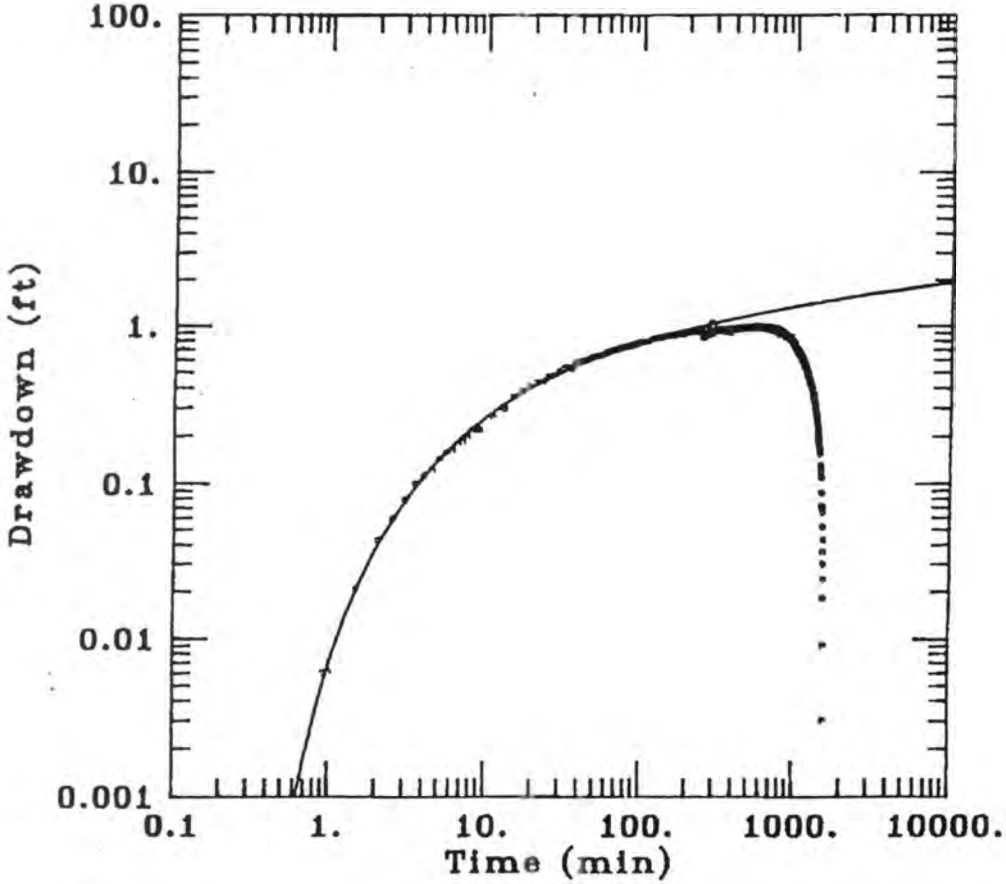
ESTIMATED PARAMETERS:
T = 26.55 ft^2/min
S = 0.01176

TEST DATA:
Q = 46.8 ft^3/min
r = 272. ft
b = 90. ft

Figure 7

GERAGHTY & MILLER	Client: MONSANTO
Project No.: NY64207	Location: SAUGET, ILLINOIS

P-9



DATA SET:
p-9.aqt
07/01/92

AQUIFER TYPE:
Unconfined
SOLUTION METHOD:
Theis
TEST DATE:
JUNE 17, 1992
TEST WELL:
TW-1
OBS. WELL:
P-9

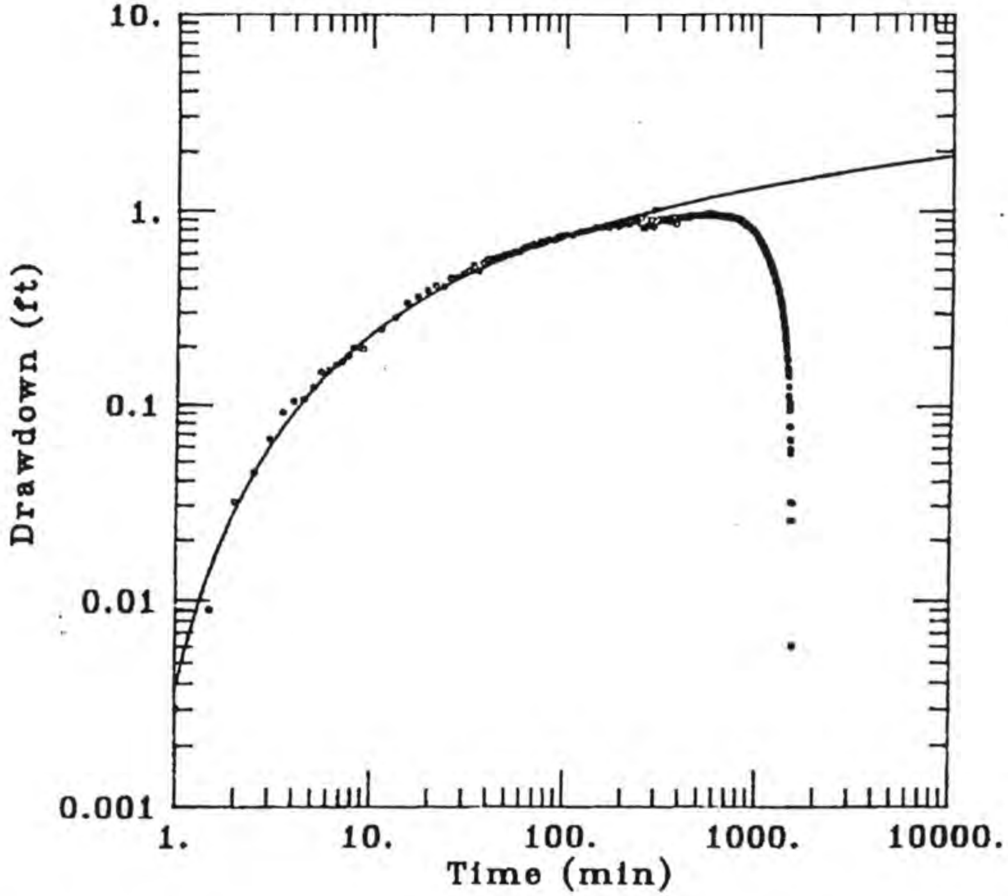
ESTIMATED PARAMETERS:
T = 15.28 ft^2/min
S = 0.01342

TEST DATA:
Q = 46.8 ft^3/min
r = 104. ft
b = 90. ft

Figure 8

GERAGHTY & MILLER	Client: MONSANTO
Project No.: NY64207	Location: SAUGET, ILLINOIS

P-10



DATA SET:
p-10.aqt
07/01/92

AQUIFER TYPE:
Unconfined

SOLUTION METHOD:
Theis

TEST DATE:
JUNE 17, 1992

TEST WELL:
TW-1

OBS. WELL:
P-10

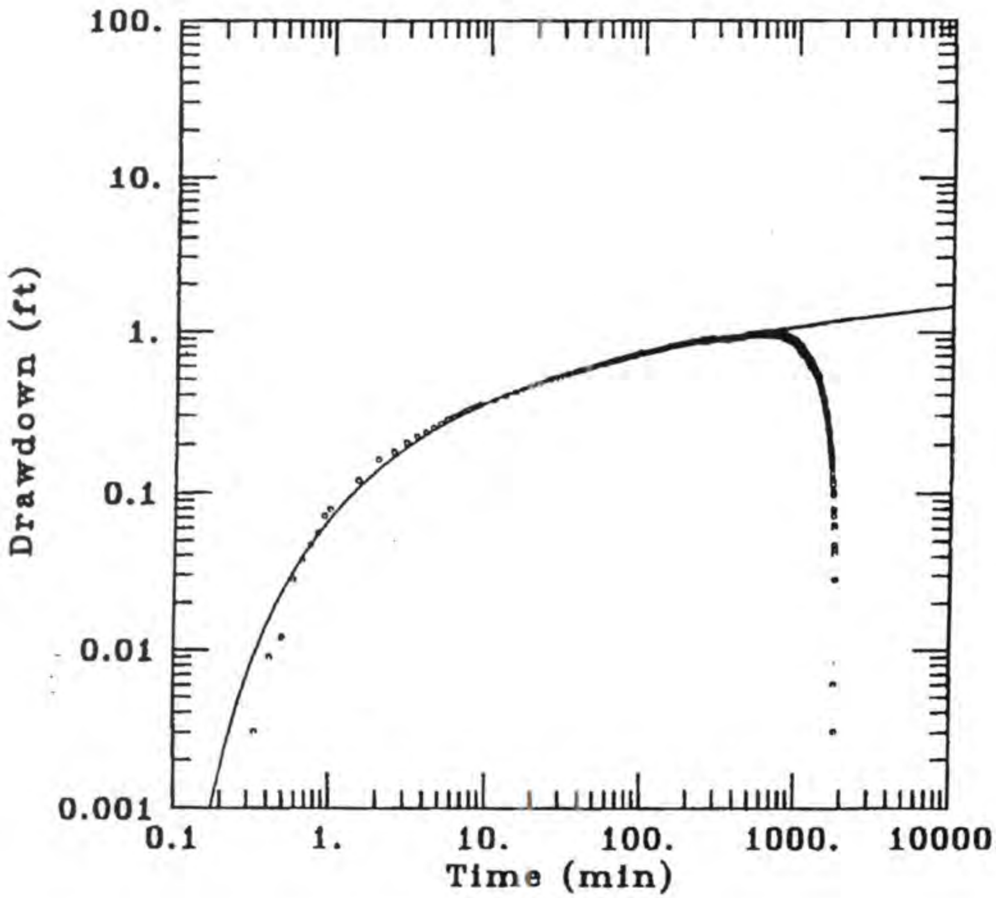
ESTIMATED PARAMETERS:
T = 15.19 ft^2/min
S = 0.008326

TEST DATA:
Q = 46.8 ft^3/min
r = 143. ft
b = 90. ft

Figure 9

G & M	Client: MONSANTO
Project No.: NY64207	Location: SAUGET, ILLINOIS

B-24C



DATA SET:
B-24C.AQT
07/08/92

AQUIFER TYPE:
Unconfined
SOLUTION METHOD:
Theis
TEST DATE:
JUNE 17, 1992
TEST WELL:
TW-1
OBS. WELL:
B-24C

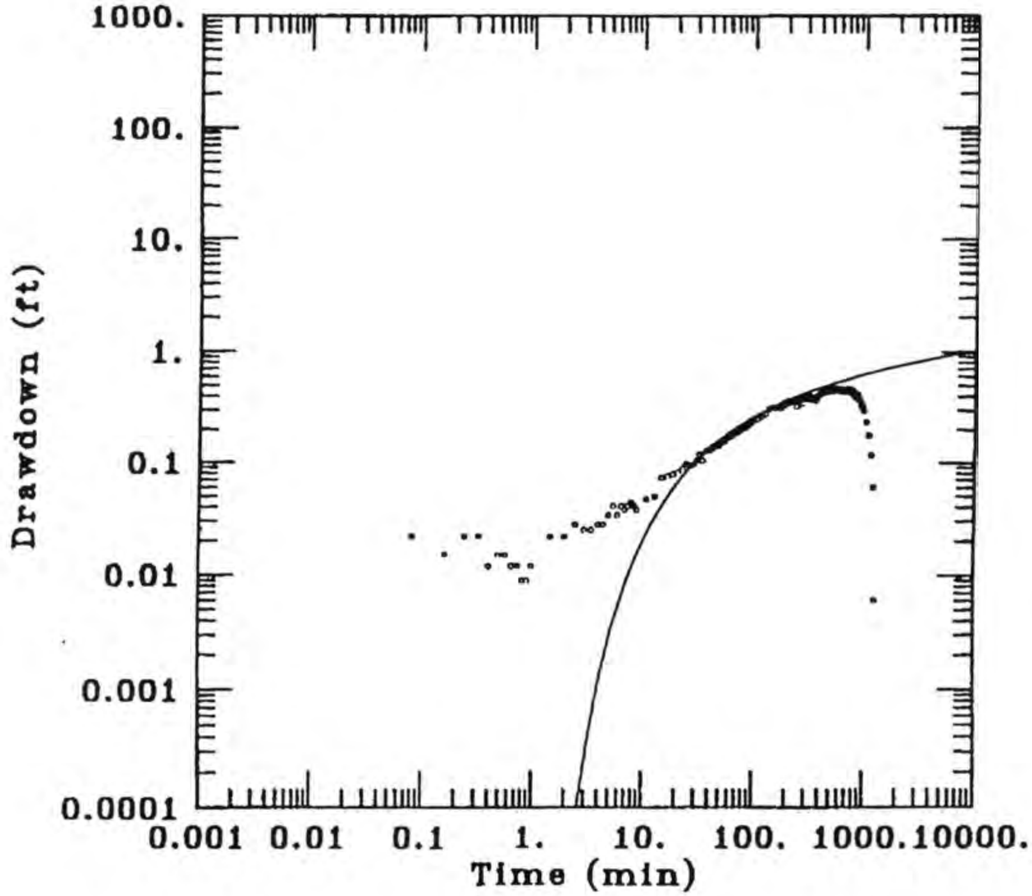
ESTIMATED PARAMETERS:
T = 23.8 ft^2/min
S = 0.004155

TEST DATA:
Q = 46.8 ft^3/min
r = 118. ft
rc = 0.5 ft
rw = 0.5 ft
b = 90. ft

Figure 10

GERAGHTY & MILLER	Client: MONSANTO
Project No.: NY64207	Location: SAUGET, ILLINOIS

B-26B



DATA SET:
B-26B.AQT
07/08/92

AQUIFER TYPE:
Unconfined

SOLUTION METHOD:
Theis

TEST DATE:
JUNE 17, 1992

TEST WELL:
TW-1

OBS. WELL:
B-26B

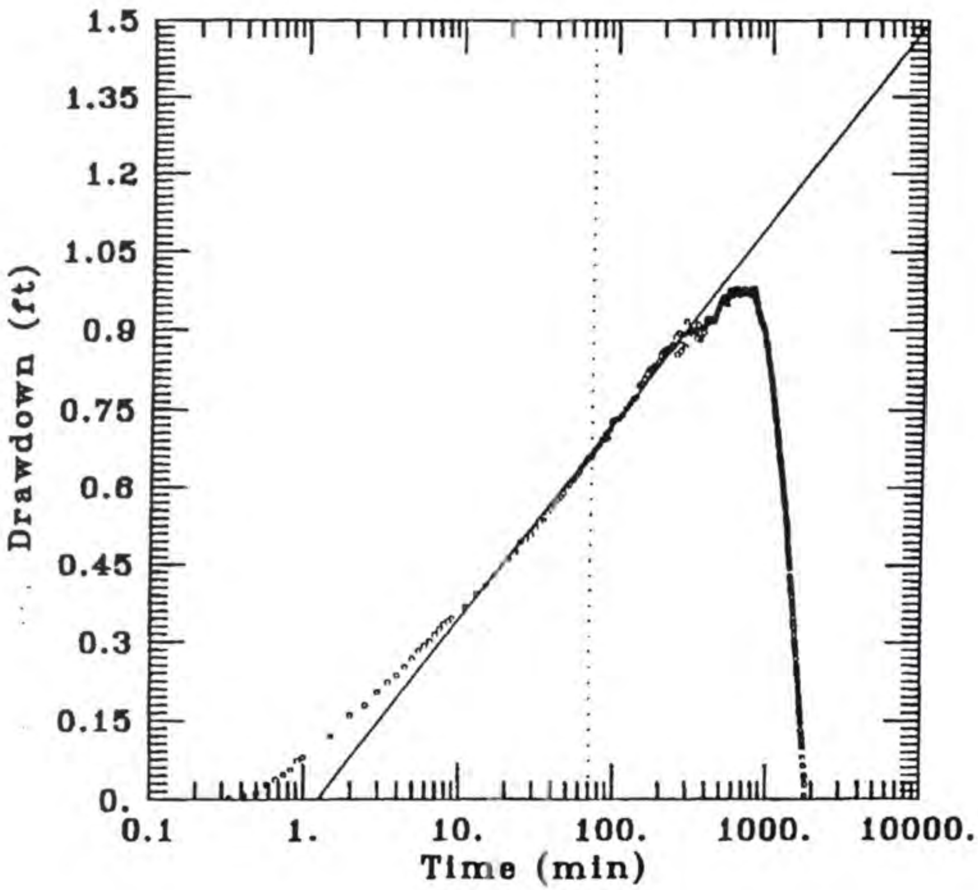
ESTIMATED PARAMETERS:
T = 22.22 ft^2/min
S = 0.006535

TEST DATA:
Q = 46.8 ft^3/min
r = 433. ft
rc = 0.5 ft
rw = 0.5 ft
b = 90. ft

Figure 11

G & M	Client: MONSANTO
Project No.: NY64207	Location: SAUGET, ILLINOIS

B-24C



DATA SET:
B-24C.AQT
07/08/92

AQUIFER TYPE:
Unconfined
SOLUTION METHOD:
Cooper-Jacob
TEST DATE:
JUNE 17, 1992
TEST WELL:
TW-1
OBS. WELL:
B-24C

ESTIMATED PARAMETERS:
T = 22.53 ft^2/min
S = 0.004504

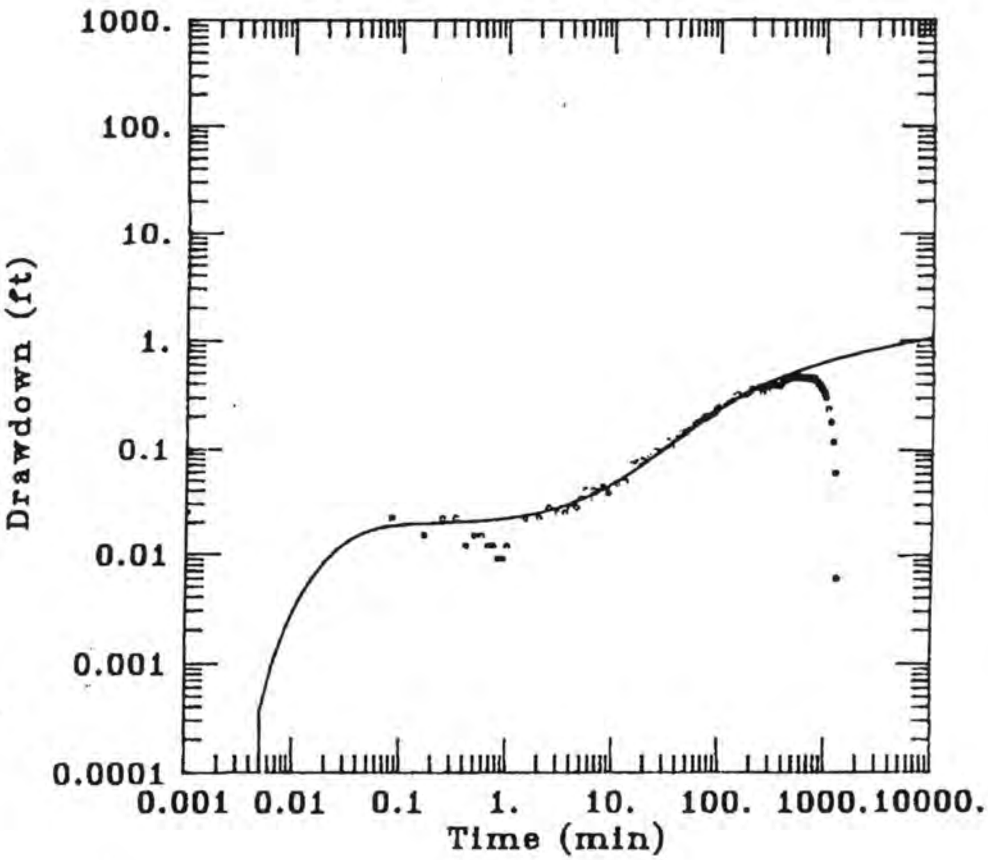
TEST DATA:
Q = 46.8 ft^3/min
r = 118. ft
rc = 0.5 ft
rw = 0.5 ft
b = 90. ft

Figure 12

GERAGHTY & MILLER | Client: MONSANTO

Project No.: NY64207 | Location: SAUGET, ILLINOIS

B-26B



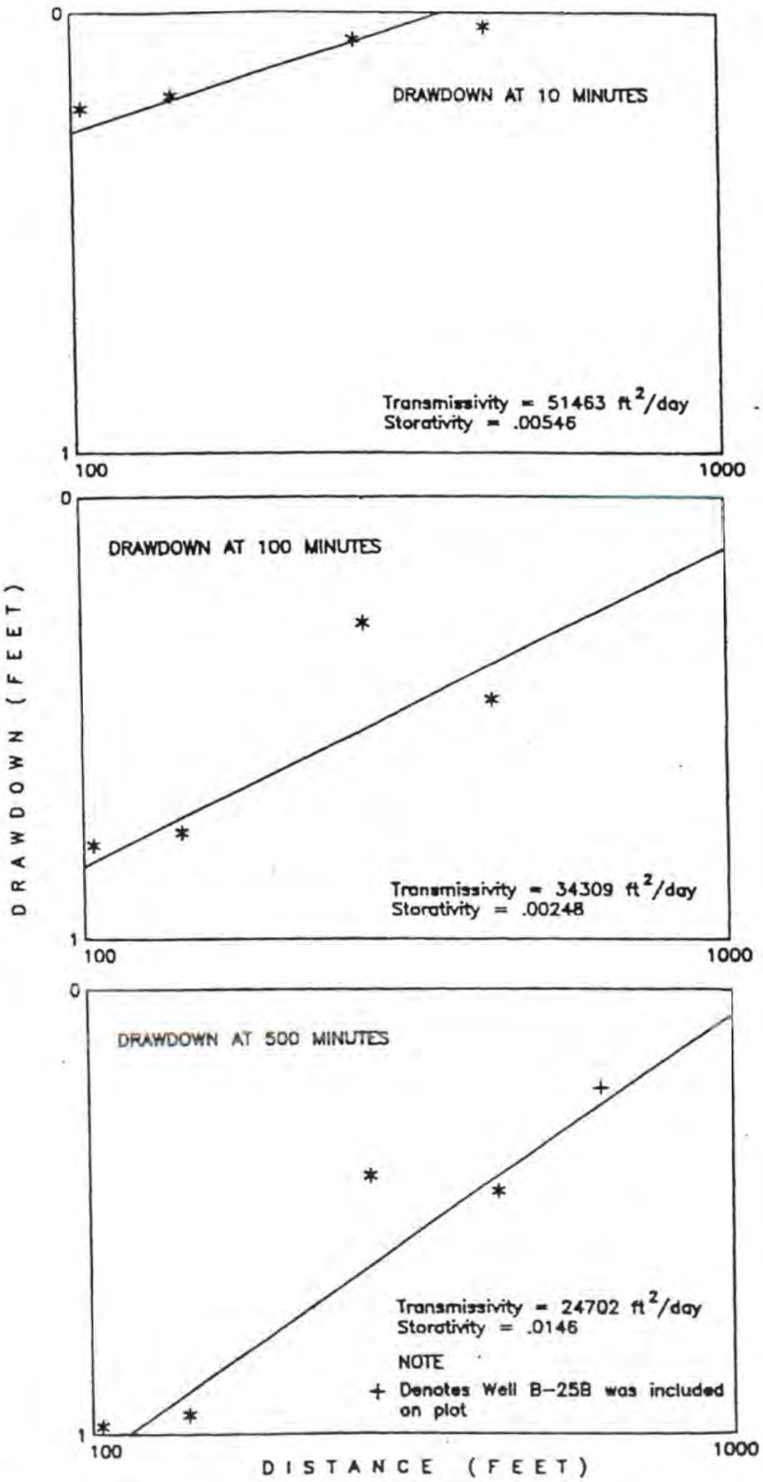
DATA SET:
b-26b.aqt
07/01/92

AQUIFER TYPE:
Unconfined
SOLUTION METHOD:
Neuman
TEST DATE:
JUNE 17, 1992
TEST WELL:
TW-1
OBS. WELL:
B-26B

ESTIMATED PARAMETERS:
T = 20.3 ft^2/min
S = 6.8199E-06
Sy = 0.007037
β = 0.8

TEST DATA:
Q = 46.8 ft^3/min
r = 433. ft
rc = 0.5 ft
rw = 0.5 ft
b = 99. ft
TEST WELL:
t.o.s. = 41.5 ft
b.o.s. = 73.5 ft
OBS. WELL:
t.o.s. = 2.4 ft
b.o.s. = 12.4 ft

Figure 13



DRAFTER: GS | APPROVED: LM | CHECKED: SF | DRAWING: IN. .08 | FILE NO.: 1558 | PRJCT NO.: NY64207 | 2-9-93 | DWG L

GERAGHTY & MILLER, INC.
Environmental Services

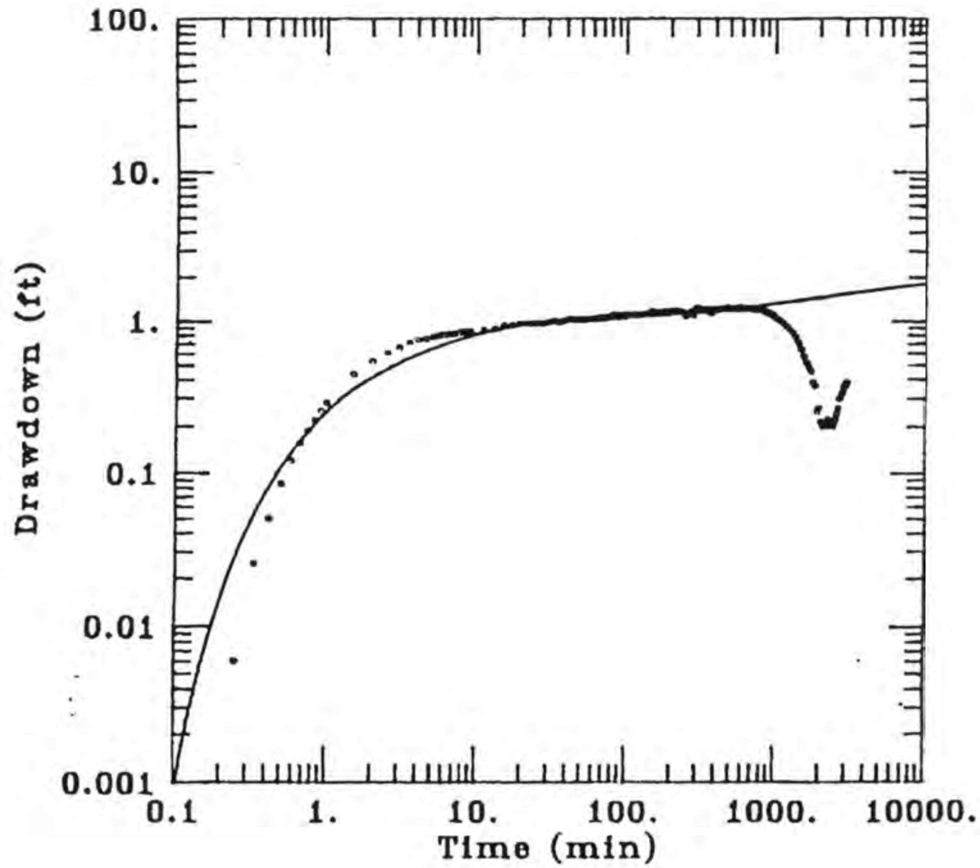
JACOB DISTANCE-DRAWDOWN PLOTS OF FOUR INTERMEDIATE ZONE WELLS AT 10, 100, AND 500 MINUTES INTO THE AQUIFER TEST

MONSANTO COMPANY
SAUGET, ILLINOIS

FIGURE 14

GERAGHTY & MILLER	Client: MONSANTO
Project No.: NY64207	Location: SAUGET, ILLINOIS

GM-56C



DATA SET:
GM-56C.AQT
06/29/92

AQUIFER TYPE:
Unconfined
SOLUTION METHOD:
Neuman
TEST DATE:
JUNE 17, 1992
TEST WELL:
TW-1
OBS. WELL:
GM-56C

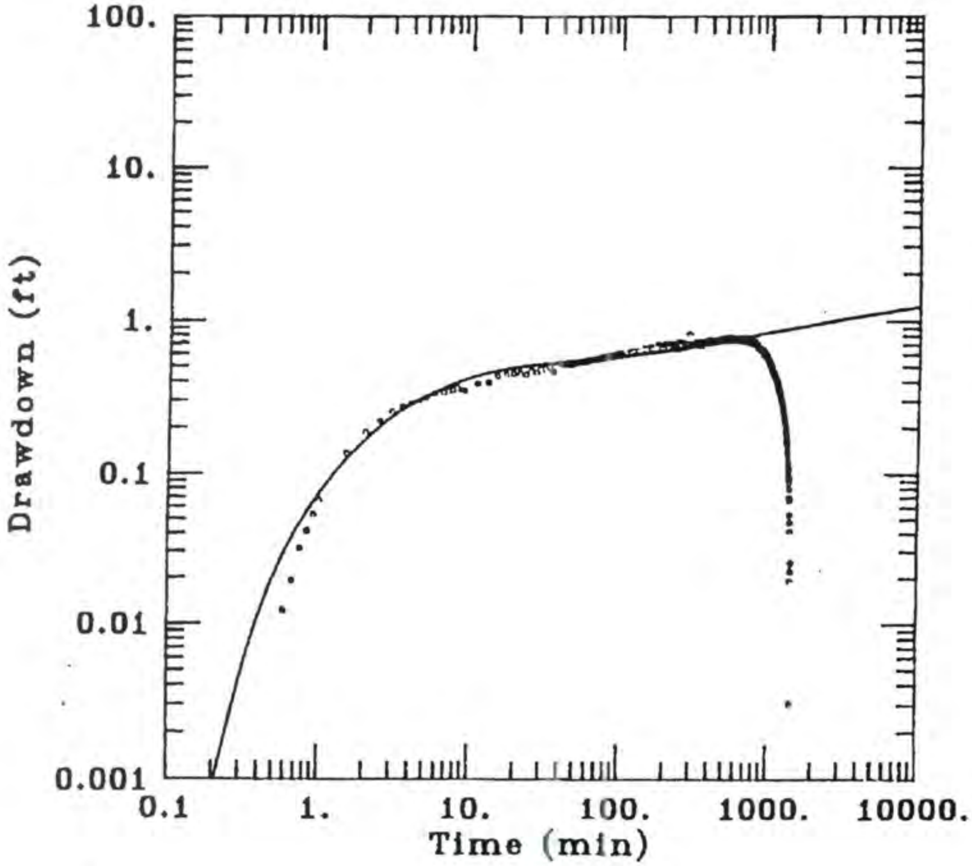
ESTIMATED PARAMETERS:
T = 16.64 ft^2/min
S = 0.001341
Sy = 0.01647
β = 0.03

TEST DATA:
Q = 46.8 ft^3/min
r = 150.5 ft
rc = 0.5 ft
rw = 0.5 ft
b = 90. ft
TEST WELL:
t.o.s. = 41.5 ft
b.o.s. = 73.6 ft
OBS. WELL:
t.o.s. = 58.1 ft
b.o.s. = 78.1 ft

Figure 15

GERAGHTY & MILLER	Client: MONSANTO
Project No.: NY64207	Location: SAUGET, ILLINOIS

GM−57C



DATA SET:
gm-57c.aqt
07/01/92

AQUIFER TYPE:
Unconfined

SOLUTION METHOD:
Neuman

TEST DATE:
JUNE 17, 1992

TEST WELL:
TW-1

OBS. WELL:
GM-57C

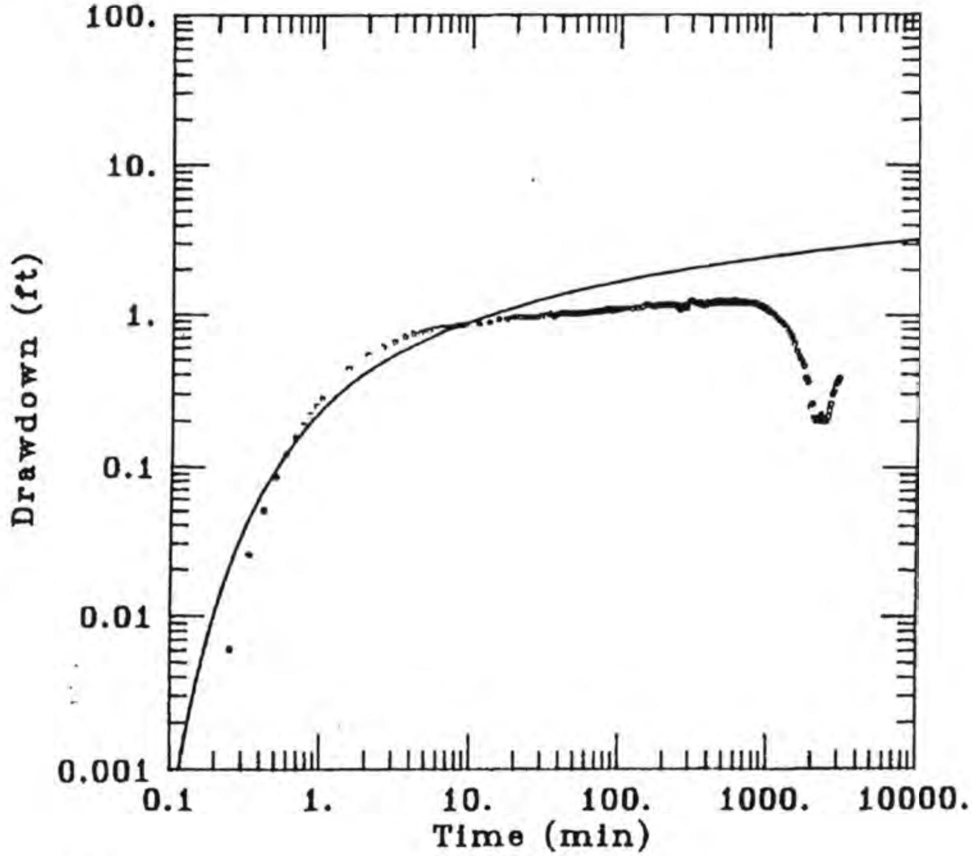
ESTIMATED PARAMETERS:
T = 20.65 ft^2/min
S = 0.0005388
Sy = 0.005517
β = 0.1

TEST DATA:
Q = 46.8 ft^3/min
r = 368. ft
rc = 0.5 ft
rw = 0.5 ft
b = 90. ft
TEST WELL:
t.o.s. = 41.5 ft
b.o.s. = 73.5 ft
OBS. WELL:
t.o.s. = 60. ft
b.o.s. = 80. ft

Figure 16

GERAGHTY & MILLER	Client: MONSANTO
Project No.: NY64207	Location: SAUGET, ILLINOIS

GM-56C



DATA SET:
gm-56c.aqt
06/25/92

AQUIFER TYPE:
Leaky
SOLUTION METHOD:
Hantush
TEST DATE:
JUNE 17, 1992
TEST WELL:
TW-1
OBS. WELL:
GM-56C

ESTIMATED PARAMETERS:
T = 10.82 ft^2/min
S = 0.001283
β = 0.0001

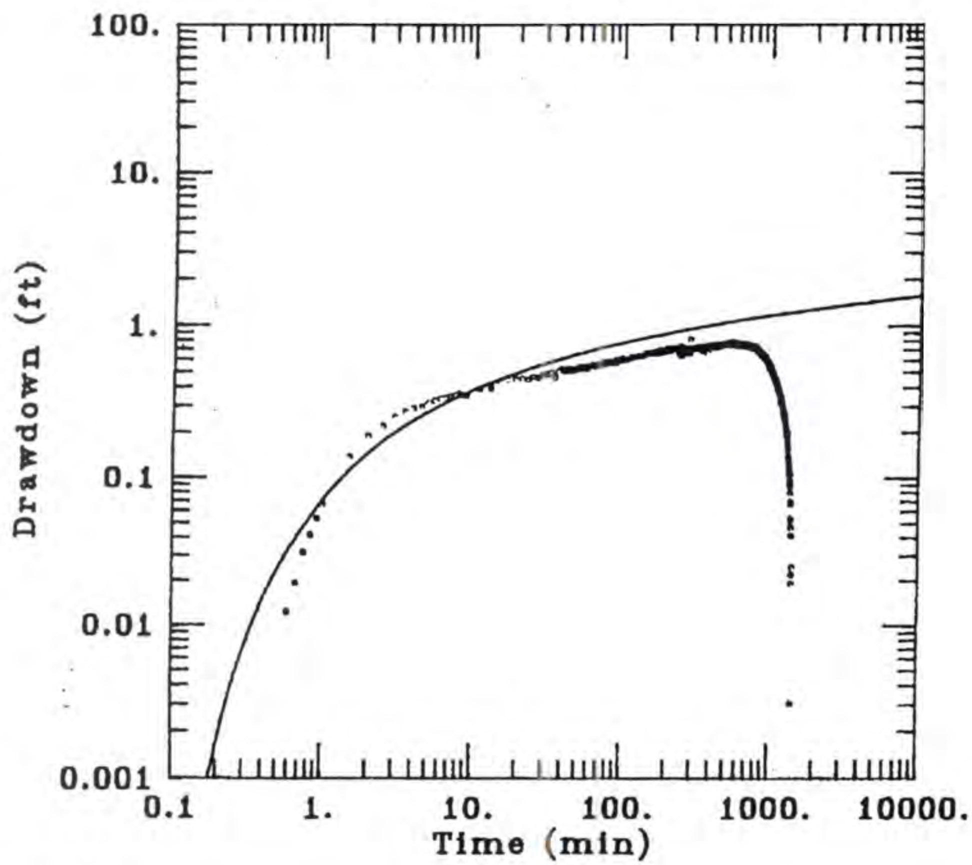
TEST DATA:
Q = 46.8 ft^3/min
r = 135. ft
rc = 0.5 ft
rw = 0.5 ft
b = 90. ft
TEST WELL:
t.o.s. = 41.5 ft
b.o.s. = 73.5 ft
OBS. WELL:
t.o.s. = 58.1 ft
b.o.s. = 78.1 ft

Figure 17

GERAGHTY & MILLER | Client: MONSANTO

Project No.: NY64207 | Location: SAUGET, ILLINOIS

GM-57C



DATA SET:
gm-57c.aqt
07/01/92

AQUIFER TYPE:
Leaky
SOLUTION METHOD:
Hantush
TEST DATE:
JUNE 17, 1992
TEST WELL:
TW-1
OBS. WELL:
GM-57C

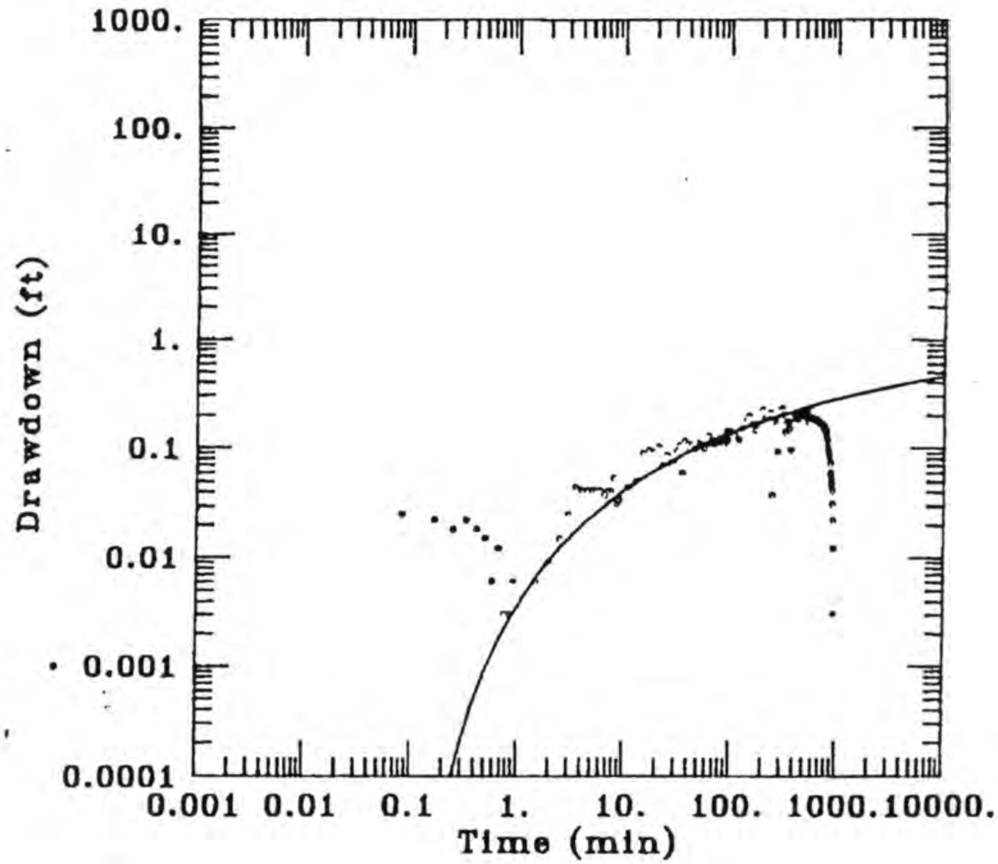
ESTIMATED PARAMETERS:
T = 21.43 ft^2/min
S = 0.0004317
β = 0.0001

TEST DATA:
Q = 46.8 ft^3/min
r = 368. ft
rc = 0.5 ft
rw = 0.5 ft
b = 90. ft
TEST WELL:
t.o.s. = 41.5 ft
b.o.s. = 73.5 ft
OBS. WELL:
t.o.s. = 60. ft
b.o.s. = 80. ft

Figure 18

GERAGHTY & MILLER	Client: MONSANTO
Project No.: NY64207	Location: SAUGET, ILLINOIS

GM-28C



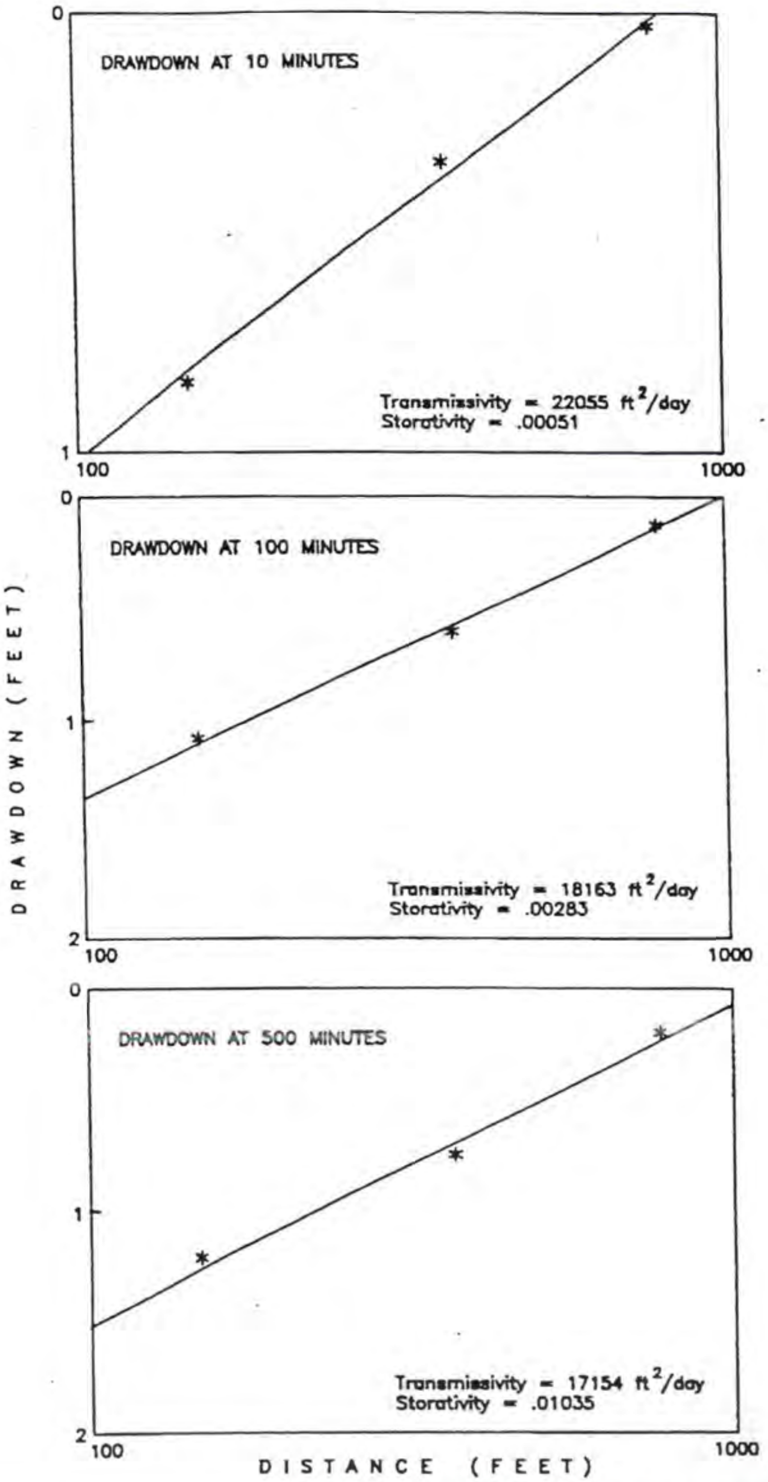
DATA SET:
gm-28c.aqt
07/01/92

AQUIFER TYPE:
Leaky
SOLUTION METHOD:
Hantush
TEST DATE:
JUNE 17, 1992
TEST WELL:
TW-1
OBS. WELL:
GM-28C

ESTIMATED PARAMETERS:
T = 21.64 ft^2/min
S = 0.0001029
β = 2.

TEST DATA:
Q = 46.8 ft^3/min
r = 772. ft
rc = 0.5 ft
rw = 0.5 ft
b = 90. ft
TEST WELL:
t.o.s. = 41.5 ft
b.o.s. = 73.5 ft
OBS. WELL:
t.o.s. = 51. ft
b.o.s. = 71. ft

Figure 19



DRAFTER: GS | APPROVED: LM | CHECKED: SF | DRAWING: DL .AC | FILE NO.: 1558 | PRJCT NO.: NY64207 | DWG . 2-9-93

GERAGHTY & MILLER, INC.
Environmental Services

JACOB DISTANCE-DRAWDOWN PLOTS OF THREE DEEP ZONE WELLS AT 10, 100, AND 500 MINUTES INTO THE AQUIFER TEST

MONSANTO COMPANY
SAUGET, ILLINOIS

FIGURE 20

DEVELOPMENT OF A
THREE-DIMENSIONAL
GROUND-WATER FLOW MODEL FOR
SAUGET SITE R,
SAUGET, ILLINOIS

February 1993

Prepared for

Monsanto Company
800 North Lindergh Boulevard
St. Louis, Missouri 63167

Prepared by

Geraghty & Miller, Inc.
125 East Bethpage Road
Plainview, New York 11803
(516) 249-7600

CONTENTS

TABLES

FIGURES

FIGURES (continued)

LM:MONSANTO SITE R MODELING REPORT\MODEL.TOC\NY642.11

DEVELOPMENT OF A THREE-DIMENSIONAL GROUND-WATER FLOW MODEL FOR SAUGET SITE R, SAUGET, ILLINOIS

1.0 INTRODUCTION

Monsanto Company retained Geraghty & Miller, Inc. to construct a three-dimensional ground-water flow model for Sauget Site R and surrounding area in Sauget, Illinois. The purpose of the project was to develop a calibrated model to simulate ground-water flow at Sauget Site R, which is shown on Figure 1. Contained in this report is the documentation of the model construction and calibration. The model was calibrated successfully to low flow conditions representing base flow to the Mississippi River using water-level data measured in November 1988. This time period represented a prolonged period of base flow conditions in the Mississippi River. The model was further tested by calibrating to high river stage conditions which occurred in November 1985.

A ground-water model is a powerful tool for analyzing current ground-water flow conditions and for predicting the impacts of remedial actions on the ground-water system. Development of an accurate model requires the integration of all available data defining the flow system. The current Sauget model incorporates all ground-water data collected through August 1992, including results from the June 1992 aquifer test conducted at Site R.

The scope of the ground-water flow modeling analysis included three main tasks: (1) data review and organization, (2) conceptual model development, and (3) model calibration. The purpose of the ground-water flow analysis was to develop a calibrated steady-state, ground-water model suitable for predicting water levels over a wide range of future conditions and potential system stresses.

The data review phase of the ground-water flow analysis examined all data pertinent to the ground-water system. In general, four fundamental types of information are required for modeling a particular hydrogeologic system: (1) geologic framework, (2) hydraulic properties, (3) water levels, and (4) fluid sources and sinks (pumping rates, recharge, etc.). The data review and organization phase resulted in development of a modeling database. This database facilitates the integration and analysis of data about the hydrogeologic system. The database forms the foundation of the conceptual model and provides the necessary information used during the model construction and calibration.

The Monsanto database includes geologic information from the extensive work Geraghty & Miller and other consultants have conducted at Sauget Site R and Monsanto's W.G. Krummrich plant in Sauget, Illinois. Water levels have been monitored biannually since 1984 and water-level recording instruments have provided continuous water-level data at nine locations in the area during that time period. Sources and sinks in the ground-water system include the Mississippi River, the Harding Ditch and associated tributaries, as well as the small lakes located to the north of the Harding Ditch.

The conceptual model, a succinct description of the important components of the ground-water system, was developed on the basis of the data review. The conceptual model formulates input data for the mathematical model by identifying initial values for hydraulic parameters. The conceptual model also guides calibration of the numerical model and aids in interpreting model results. The conceptual model of the ground-water flow system is presented in the next section.

After developing a conceptual understanding of the ground-water flow system, the numerical model was constructed. Model construction consisted of discretizing the flow system into rectangular blocks, assigning aquifer properties to each block, and estimating ground-water sources and sinks. Model data sets were constructed for the USGS Modular Three-Dimensional Flow Model, also know as MODFLOW (McDonald and Harbaugh 1988). MODFLOW is a well-accepted public domain software package for modeling three-dimensional ground-water flow.

Model calibration refers to the process of adjusting hydraulic parameters to obtain a reasonable match between water levels measured in the field and water levels calculated by the model. The Site R model was calibrated to water levels measured in November 1988 (base flow conditions) and to water levels measured during a flood event in November 1985. The 1988 calibration is termed a steady-state calibration and represents base flow conditions in the ground-water basin. The 1985 calibration was performed transiently to a short-term flood event where ground-water gradients were reversed in the vicinity of the Mississippi River and Sauget Site R. Quantitative or statistical comparisons were made between the site water-level data and model-computed heads for the steady-state calibration, while only a qualitative comparison was made for the transient calibration. The transient calibration was evaluated qualitatively because only one set of measurements was available for a large transient event. Thus, there was more uncertainty involved in the transient analysis than in the steady-state calibration.

2.0 CONCEPTUAL MODEL

2.1 GEOLOGY

Sauget Site R and the surrounding area included in the ground-water flow model are located in southwestern Illinois on the flood plain of the Mississippi River, named the American Bottoms (Figure 1). The flood plain is surrounded by bedrock bluffs on the eastern boundary of the model and across the Mississippi on the western boundary of the model. The flood plain contains unconsolidated deposits composed of recent alluvium (Cahokia Alluvium) which overlies glacial material (Henry Formation). Underlying the unconsolidated deposits is Mississippian and Pennsylvanian limestone and dolomite with lesser amounts of sandstone and shale. The average thickness of the unconsolidated material across the model area is approximately 130 ft.

To simplify the flow system and thus the model, the unconsolidated deposits were categorized into three hydrogeologic zones. They are as follows: the water-table (shallow) zone (Layer 1), intermediate zone (Layer 2), and deep zone (Layer 3). The following discussion will be limited to these three zones. The bedrock is not included in the model because it is not an important aquifer due to low permeability. Although the water-table, intermediate, and deep zones have variable thicknesses, a uniform thickness and depth interval was assigned to each subsurface zone for the purpose of modeling. The thickness of each zone is provided later in this section. These zone distinctions are based on the differences in subsurface lithologic conditions. Wells used to guide the modeling effort are shown on the site location map (Figure 2). Delineation of the three zones and their relationships to the layers are shown on the generalized east-west cross section found on Figure 3. The cross section lies in the western portion of the model area, which has good geologic control due to extensive drilling by Geraghty & Miller and others at Sauget Site R and at the W.G. Krummrich plant. The geology is fairly uniform throughout the model area and, therefore, only an east-west cross section is necessary.

The water-table zone consists of the Cahokia Alluvium (recent deposits), which is an unconsolidated, fine-grained silty sand. For the purposes of the model, the layer is considered to be 30 ft thick, starting at the water table and continuing down to the medium sand deposits of the Henry Formation (bottom elevation of the layer is 365 ft msl). The cross section (Figure 3) shows Layer 1 to be a low permeability zone with fine-grained silty sand deposits predominating.

The intermediate zone is much coarser than the overlying water-table zone. This zone contains medium-grained sand representing the upper portion of the Henry Formation, a Wisconsinan glacial outwash in the form of valley-train deposits. Valley train deposits are long narrow bodies of outwash, deposited by meltwater streams far beyond the terminal moraine and confined within the walls of a valley. The Henry Formation is characterized by medium to coarse sand becoming coarser with depth. Thickness of model Layer 2 is 45 ft. This corresponds well to the range of thickness in the cross section (Figure 3).

Below the intermediate zone is the deep zone (Layer 3) which is marked by coarser deposits of the lower portion of the Henry Formation. In some areas, till and/or boulder zones were encountered 10 to 15 ft above the bedrock. The coarser deposits are delineated by the model to be 35 ft thick (Figure 3).

2.2 PREVIOUS NUMERICAL MODELING STUDY

A modeling study of the entire American Bottoms ground-water flow system was conducted by the Illinois State Water Survey Division (Ritchey et al. 1984). The purpose of this study was to conduct a detailed investigation of the flow systems in the area. Then current hydrologic data pertaining to the area were compiled, a computer model was developed to simulate the movement of the ground water, ground-water levels in the area were analyzed, and future ground-water levels were predicted. Documentation of the model, including a user guide, was also included.

The compilation of hydrogeologic information included the distribution of pumpage in the area including the major and minor pumping centers and pumpage from wells adjacent to the Mississippi River. A series of hydrographs from the years 1940 to 1981 were plotted and included in the report.

The ground-water model used was a modified form of the Illinois State Water Survey aquifer model (Prickett and Lonnquist 1971). Modifications were made to incorporate river stage and precipitation. The model was calibrated by history matching two 5-year periods with constant 1-month time steps. Hydrographs of actual and simulated water levels of ten observation wells and the nearest model well for the two 5-year periods were presented. The model was found to consistently calculate water levels within 2 ft of the actual measured water level within a specified area of interest.

Ground-water levels were evaluated with the aid of ground-water level exceedance plots. Ground-water level exceedance probability plots were constructed for ten model wells by compiling the maximum yearly water levels from monthly simulated values. Plots were based on simulation of the 30-year period from 1951-1980. Mississippi River stage was also simulated during the 30-year period from 1951-1980.

2.3 HYDRAULIC PROPERTIES

In 1986, Geraghty & Miller compiled hydraulic properties that were determined from aquifer tests and slug tests run by Geraghty & Miller and other consultants (Geraghty & Miller, 1986b). These data are listed in Table 1. In general, the hydraulic conductivities of the intermediate and deep zones are much greater than that of the shallow water-table zone.

A detailed aquifer test was conducted by Geraghty & Miller in June 1992. The results from this test indicate that the intermediate and deep zones have approximately equal permeability with an average of 315 ft/d (Table 2). The storage coefficient was calculated to be 0.007. The overall transmissivity of the combined intermediate and deep zones was found to be about 30,000 ft^2/d which was used in the model, and 15,000 ft^2/d was applied to each zone. The construction of the model is described in Section 3.0.

2.4 RECHARGE

Average annual rainfall in the Sauget area is approximately 34 inches. Based on a 30-year average (1951 to 1980) for precipitation in the Sauget area, 13 inches of precipitation are estimated to infiltrate into the ground as recharge to the aquifer system. The calibrated steady-state model represents base flow conditions, so a lower value of recharge was used (about 9 inches/year).

3.0 GROUND-WATER FLOW MODEL CONSTRUCTION

3.1 CODE SELECTION

Ground-water flow in the Sauget area was modeled with the USGS Modular Three-Dimensional Finite-Difference Ground-Water Flow Model (McDonald and Harbaugh 1988), also known as MODFLOW. The three-dimensional capabilities of this code are appropriate for the proper treatment of the vertically variable hydrostratigraphy (three distinct aquifer zones) and boundary conditions at the study site. MODFLOW is also well documented, publicly available, and generally accepted within the scientific community.

Prior to the simulation of ground-water flow at Sauget Site R and vicinity using MODFLOW, the model was calibrated using an automatic (inverse) parameter estimation algorithm incorporated into the MODFLOW code by Duffield (1988). The inverse algorithm systematically selects a set of user-specified hydraulic parameter values that provide a least-squares match between observed and calculated water levels. Hydraulic parameters estimated in the Sauget model include: (1) hydraulic conductivity in the water-table zone (Layer 1), (2) vertical leakance across the water-table/intermediate and intermediate/deep boundaries, (3) vertical leakance of the Mississippi River bottom sediments, and (4) precipitation recharge. The transmissivity of the intermediate and deep zones (Layers 2 and 3, respectively) was maintained at the value estimated from the June 1992 aquifer test and was not changed during calibration.

3.2 MODEL CONFIGURATION

3.2.1 Discretization

The Monsanto model includes Sauget Site R, the entire W.G. Krummrich facility, and a large amount of the surrounding area, as shown on Figure 4. The model grid covers 58 square miles around the Sauget area with an east-west dimension of 44,000 ft and a north-south dimension of 37,000 ft. The model is much larger than the area of interest to incorporate regional ground-water flow effects at the site scale. The model extends to the bedrock bluffs east and west of the site (across the Mississippi River) and to Old Prairie Dupont Creek south of the site. The northern boundary of the model coincides with the center of a pumping cone of depression caused by dewatering efforts near the Poplar Street Bridge.

In the finite-difference modeling technique used in MODFLOW, the aquifer is divided into rectangular regions known as cells. The maximum cell dimension in the Sauget model is 1,000 ft. These large cells were placed away from the areas of interest. Finer grid spacings were used near Sauget Site R and the W.G. Krummrich Plant. The smallest cells measure 250 ft on a side. A portion of the finite-difference grid covering Site R and the Krummrich Plant is shown on Figure 5. This figure is provided to illustrate the finer detail used to model these areas.

The model contains three layers representing the Cahokia Alluvium (Layer 1) and Henry Formation (Layers 2 and 3). The upper model layer is unconfined and the lower two layers are semiconfined, although there are no continuous aquitards separating any of the model layers. The flow of ground water between model layers is represented in the model using a leakance term. The leakance term incorporates the lower vertical permeability characteristic of most glaciofluvial deposits to retard the movement of ground water between the three aquifer zones.

3.2.2 Boundary Conditions

To represent the variety of physical boundaries to the aquifer system in the Sauget area, several types of boundary conditions were prescribed in the ground-water flow model. A boundary condition is a numerical representation of a physical boundary or process effecting the aquifer system. These physical boundaries and processes include: (1) surface-water bodies and streams (Mississippi River and the lakes northeast of the site), (2) production wells in the Sauget area, (3) the vertical and lateral limit of the unconsolidated aquifer system, and (4) precipitation recharge.

Two primary types of numerical boundary conditions were used in the Monsanto model to represent these physical boundaries to the system. The model boundary conditions are termed constant head and flux boundaries. A third type of boundary condition, called a head-dependent flux boundary condition, was not employed in this model. The latter may be used to represent drains, for example, but there are no such features in the area.

For the purposes of calibration, constant head boundaries in the upper model layer were used to represent all surface water features including the Mississippi River, the Harding Ditch, and other small streams. In a constant head boundary cell, the ground-water level is fixed at a specified point for the duration of the simulation. This provides a continuous source or sink for ground water in the surrounding aquifer. The water-level value specified in a constant head which represents a surface stream is equal to the water elevation on the stream. A river stage of 381 ft msl was estimated for the Mississippi River from the gauging station at St. Louis, Market Street (Mile 179.6). Elevations for the remaining surface-water bodies were estimated from USGS topographic maps of the area.

A constant flux boundary condition represents a continuous and constant inflow or outflow of water within a model cell. Rather than specifying a constant water elevation, a constant discharge or recharge rate is used. Constant flux boundary conditions typically represent wells, recharge, or areas of no ground-water flow (the flux is zero). The latter are termed no-flow boundaries. Boundary conditions in Layer 1 are shown on Figure 5. The outer edge of cells on Figure 5 are assumed to be no-flow boundaries, except where specified as another type of boundary condition.

Constant flux boundary conditions were used in the model to represent: (1) recharge from precipitation, (2) production wells north of the site, and (3) the limit of the unconsolidated deposits (no-flow boundaries). A special form of no-flow boundary was used to represent the northern boundary of the model in all layers and the southern boundary in Layers 2 and 3. These boundaries were selected so as to be parallel with the regional ground-water flow directions. In this manner, the ground-water flow lines (or stream tubes) represent the model boundary. In theory, ground water does not flow across stream lines, and thus a no-flow boundary is formed.

The northern boundary was also selected to bisect the cone of depression surrounding a production center. This boundary takes advantage of the symmetry of the cone of depression and uses streamlines entering the production zone from the east and west. Thus, only half of the cone of depression around the pumping center is simulated. Consequently, only half of the pumping rate for these wells was used in the model.

The remaining no-flow boundaries included the eastern and western boundaries in all layers and the base of the model (bedrock surface). The eastern and western boundaries represent the bedrock bluffs as shown on Figure 4. It is assumed that the volume of water entering or leaving the unconsolidated aquifer system from the bedrock is insignificant compared to the volume of water entering from precipitation and induced leakage from the river.

Three discrete zones of recharge were defined in the model in Layer 1. The primary recharge zone covers most of the model and received 8.8 inches per year (in/yr)($2.0x10^{-3}$ ft/day). A second recharge zone was used to simulate an anomalous ground-water mound in the W.G. Krummrich Plant. This zone received 370 in/yr ($8.4x10^{-2}$ ft/day). The third recharge zone represents the clay cap which was installed over the landfill at Site R, adjacent to the Mississippi River. Recharge zone No. 3 received 2.63 in/yr ($6.00x10^{-4}$ ft/day). The location of these zones is shown on Figure 6.

The model recharge rate of 8.8 in/yr is below the average of 13 in/yr, as discussed in Section 2.5. The lower rate was used to simulate base flow conditions during dry periods of the year. The model was calibrated to the period around November 1988 when the Mississippi River was at a relatively constant but low stage.

3.3 AQUIFER PARAMETERS

Aquifer parameters such as hydraulic conductivity, transmissivity, and vertical leakance were assigned to each cell in the model using the concept of parameter zonation. This philosophy of modeling specifies several discrete values of each parameter which are assigned to groups (zones) of cells. Aquifer properties defined in the model include: (1) hydraulic conductivity of Layer 1, (2) transmissivity of Layers 2 and 3, (3) vertical leakance between model layers, and (4) bottom elevation for Layer 1.

Layer 1 was divided into three separate hydraulic conductivity zones having values of 6.2, 1.0, and 0.4 ft per day (ft/day). These values were determined during the calibration process. These hydraulic conductivity zones are referred to as Zones 1, 4, and 5, respectively. Zone 1 represents the hydraulic conductivity of the Cahokia Alluvium, Zone 4 represents the hydraulic conductivity of the landfill, and Zone 5 represents the bottom sediments of the Mississippi River. The distribution of hydraulic conductivity zones in Layer 1 is shown on Figure 7.

The calibrated values of hydraulic conductivity in Layer 1 for Zones 1, 4, and 5 fall within the range of field measurements for the Cahokia Alluvium (0.25 to 17.01 ft/day). Hydraulic conductivity Zone 1 (6.2 ft/day) represents average conditions across Layer 1, the low permeability layer. The value given by the model is very close to the average field hydraulic conductivity (4.42 ft/day) which is the range of silty sand. Hydraulic conductivity Zone 4 (1.0 ft/day) represents the filled portion of Sauget Site R which is estimated to be less permeable than the surrounding area due to the reworked nature of the landfill material. Zone 5 has a hydraulic conductivity of 0.4 which represents the Mississippi River bottom sediments, which are finer grained than the Cahokia Alluvium.

The transmissivity of the Henry Formation was defined in two zones (2 and 3 in the model). Zone 2 represents Layer 2 which has a transmissivity of 15,000 ft^2/day. Zone 3 represents Layer 3 which also has a transmissivity of 15,000 ft^2/day. Both Zones 2 and 3 are part of the glacial Henry Formation. The transmissivity values used in layers 2 and 3 were derived from the results of the June 1992 aquifer test. These values were not refined during calibration.

The leakage of water between the three layers was treated using a leakance term. The leakance term was calculated using the vertical permeabilities and the thickness of the layers. Five leakance zones were determined during the calibration. The vertical leakances between Layers 1 and 2 are 0.0063 day^{-1} (Zone 1, Water-table Layer/Intermediate Layer), 1.0×10^{-9} day^{-1} (Zone 3, water-table layer/intermediate layer in the landfill area), and 0.42 day^{-1} (Zone 4, Mississippi River/Intermediate Layer). Figure 8 depicts the vertical leakance zones in Layer 1. The leakance between Layer 2 and 3 has a value of 1.00 day^{-1} (Zone 2, Intermediate Layer/Deep Layer). The leakance for Zone 5 in the intermediate layer/deep layer (Mississippi River) is 0.25 day^{-1}.

A summary of the hydraulic parameter zones and their model calibrated values are shown in Table 3, which also includes the recharge values discussed in the previous section. All values were estimated using the automatic calibration procedure which is described in the next section.

4.0 STEADY-STATE MODEL CALIBRATION

4.1 CALIBRATION TECHNIQUE

A ground-water model is calibrated by adjusting aquifer properties (hydraulic conductivity, transmissivity, and vertical leakance) and boundary conditions within reasonable limits to obtain an acceptable match between observed and calculated ground-water levels. The reasonable limits within which parameters may be varied is determined by field testing and by values reported in the scientific literature. Many single-well aquifer tests and slug tests were used to set reasonable limits for hydraulic conductivity in the Sauget area. An acceptable match between water levels measured in the field and those calculated by the model is determined through graphical and statistical analysis of residuals. A residual is the difference between observed water levels (field measurements) and water levels calculated by the model.

The model was calibrated using a nonlinear least-squares technique known as the Marquardt Algorithm (Marquardt 1963). This technique is often referred to as "automatic calibration" or inverse modeling. Inverse techniques determine optimum aquifer parameter values for a given model configuration (grid spacing and boundary conditions) which provide the best statistical calibration. The calibration for the model was arrived at through an iterative procedure involving inverse model runs and subsequent redefinition of aquifer parameter zones and boundary conditions. Parameter values for the final calibrated model were described in the previous section.

Two types of calibrations were performed on the Sauget model. The first step consisted of calibrating the model to base flow (steady-state) conditions in the Mississippi River. The steady-state calibration was performed by comparing model-calculated water levels to those measured in the field during November 1988. This period represents a prolonged base flow period. The second calibration compared model calculations to a flood event in the Mississippi River in November 1985. The latter was a transient calibration which is discussed in Section 5.0.

4.2 CALIBRATION TARGETS

A critical component of any model calibration is a set of measured ground-water levels to compare with model calculations. These observed or measured ground-water levels are known as calibration targets. The goal in selecting calibration targets is to define a set of targets that are reliable and well distributed throughout the area of the model.

Calibration targets were selected for the model using a three-step procedure. In the first step, the November and December water levels for the years 1984 through 1988 were compiled to chose the year that would most closely represent steady state ground-water flow conditions in the area. The months of November and December were chosen because they are typically closest to base flow conditions in the Mississippi River. The standard deviation for each of the wells was also computed to assess the variability in water level measurements. Water levels from 1988 were chosen during the first phase of target selection because of prolonged base flow conditions in the Mississippi River which imparted a low standard deviation in water level measurements.

During the second phase of target selection, wells with a 1988 reading and a low standard deviation were included in the list of targets. Wells exhibiting a large standard deviation (> 3 ft) were not included in the list of targets; however, it was necessary to choose some wells near the Mississippi River with a high standard deviation. The high standard deviation is due to the extreme fluctuation in water levels near the Mississippi River because of the river's variation over time. Most of the standard deviations away from the river were less than 3 ft and near the river the deviation was approximately 6 ft. The 1988 readings were chosen because these measurements were made during a prolonged period of base flow conditions in the Mississippi River.

During the third phase of target selection, clusters of wells were reduced in number. Many of the wells are closely grouped around the landfill, for example. In order not to significantly bias the automatic calibration procedure, not all wells around the landfill were used in the calibration. Wells were chosen to provide an even distribution over the study area. Using this three-step approach, 69 target wells were chosen from a total of 164 wells. The water-table zone (Layer 1) contains the greatest number of calibration targets (30). The calibration targets in the intermediate (23) and deep (16) zones are fewer in number, but well distributed. The locations of calibration targets within the model are shown on Figures 9 through 11. These wells are also summarized in Table 4.

4.3 STEADY-STATE CALIBRATION RESULTS

One of the most important parameters used in evaluating a calibration is the residual. A residual is calculated for each calibration target by subtracting the model-calculated water level from the observed water level. A residual near zero signifies a close match between the model and observed field conditions. The sign of the residual, positive or negative, is just as important as the magnitude of the residual. Negative residuals occur where the model-calculated water levels are higher than observed. Conversely, positive residuals indicate that the model-calculated water levels are too low.

In discussing the quality of a model calibration, the following criteria must be considered: (1) the average of all residuals (residual mean) should be close to zero; (2) the variation in residuals (residual standard deviation) should be low; (3) the distribution of residuals within the model should be random; and (4) the flow patterns predicted by the model should match field observations. Most of these factors are subjective; however, all must be evaluated when determining the quality of a calibration.

All criteria listed above were satisfied in the model calibration. The residual mean (0.03 ft) was close to zero. The residual standard deviation (1.04 ft) is very low. The residuals are fairly well distributed and ground-water flow directions match field observations. Flow is toward the Mississippi River in all three layers with ground-water mounding in Layer 1 at the landfill and W.G. Krummrich plant. Figures 9 through 11 illustrate the potentiometric surfaces for the three model layers in the vicinity of Site R.

A statistical analysis of residuals quantifies the match between the simulated water levels and actual water-level measurements. The two important statistics discussed above include the residual mean and the residual standard deviation. For good calibration, the residual mean should be close to zero. This implies that positive residuals (areas where the model water levels are too low) and negative residuals (model water levels are higher than observed) are equally balanced within the model domain. In the model, the residual mean is 0.03 ft. In addition to a residual mean close to zero, the residual standard deviation should be low. The model residual standard deviation was 1.04 ft. This means that most model residuals are in error by no more than 1.04 ft. In fact, 27 of the 69 residuals are less than 0.5 ft. The residual standard deviation should also be much less than the total change in head across the site. In this case, the total water-level change across the modeled area is about 23 ft. The residual standard deviation is less than five percent of this number. Residuals for each well are listed in Table 4.

The next test of a good calibration is the spatial distribution of residuals. There are two ways of looking at spatial distribution. The first involves plotting the observed versus calculated water levels. In a perfect calibration, the calculated water levels would equal the observed water levels. The scatter of actual residuals around this perfect line is a graphical means of evaluating spatial distribution of residuals. Such a plot is presented in Figure 12. This plot illustrates that residuals at high and low points in the flow system have a random error of $\pm$ 1.0 ft. That is, there is an even scatter among the residuals and the errors are evenly distributed between high and low water levels.

The second type of spatial analysis involves plotting the residuals on a site map. Positive or negative residuals should not cluster in any area, i.e., they should be randomly distributed. Figures 9 through 11 show the residuals in Layers 1 through 3 for the areas near Site R and the W.G. Krummrich plant. There are no wells and associated residuals located outside the area displayed by Figures 9 through 11.

Residuals in Layer 1 are well distributed around Site R, however, there is minor clustering of negative residuals around the ground-water mound located in the W.G. Krummrich plant, and the overall distribution of residuals is slightly biased toward higher water levels. Layer 1 also has a number of high residuals located in the landfill. This is due to the destabilizing effect of the Mississippi River on water levels.

5.0 TRANSIENT CALIBRATION RESULTS

The steady-state calibration discussed in the previous section compared model-calculated results to water levels measured in November 1988. This calibration represents average base-flow conditions in the Mississippi River. In order to demonstrate that the ground-water flow model constructed for Sauget is valid for higher water-level events as well, a transient calibration was also performed.

The transient calibration compared model-calculated water levels to those measured in November 1985 when the Mississippi River was at a much higher stage than in November 1988. The Mississippi River stage used in the transient calibration was 410 ft msl, compared to a stage of 381 ft msl used in the steady-state calibration. The November 1985 water levels are contoured in Figures 13 through 15 for the shallow, intermediate, and deep zones, respectively.

The transient calibration differed from the steady-state calibration in that ground-water levels in the aquifer were not at equilibrium. The Mississippi River was rising for about 1 week prior to the ground-water level measurements. Consequently, the ground-water levels were also still rising. To simulate these conditions, the water-level distribution calculated by the steady-state model was used as initial conditions in the transient calibration. Next, the Mississippi River stage was increased to 410 ft msl. This was the river stage reached just prior to the round of ground-water level measurements. The model was then run for 7 days and the model-calculated heads were contoured.

Only a qualitative comparison was made between model-calculated heads and observed heads because only one round of water-level measurements (November 1985) were available for comparison during a period when high river stage lasted for several weeks. In an ideal transient calibration, water levels are available at numerous times for comparison with the model results.

The model configuration for the transient calibration was identical to the steady-state model, with two exceptions: (1) a uniform storage coefficient was assumed in each layer (no storage coefficient is necessary in a steady-state model), and (2) the recharge rate was increased 10 percent because there was a significant amount of precipitation during the week prior to the water-level measurements. The storage coefficients were adjusted during the calibration to obtain a qualitative match between the observed and calculated water levels. The final storage coefficients were 0.1 in Layer 1, and 0.03 in Layers 2 and 3. These storage values are close to those obtained from pumping test analyses (0.07, 0.04 and 0.09) as discussed in Section 2.

The final calibrated ground-water levels simulated in the transient model one week after raising the Mississippi River level are presented in Figures 16 through 18 for model Layers 1 through 3, respectively. Both the model-generated figures and those contoured from observed data (Figures 22 to 24) show a reversal in ground-water flow directions near the Mississippi River. During this time frame, ground water flowed away from the river into the aquifer. A point of converging ground-water flow is clearly identified between the Krummrich Plant and Site R. This reversal in gradient near the river occurs in all three aquifer zones. In addition to the reversal in gradient, both model results and observed water levels increased to levels above 400 ft msl between Site R and the Mississippi River.

The two methods used to calibrate the model each clearly illustrate that the numerical ground-water flow model accurately represents the aquifer system at Site R and its vicinity for both high and low flow conditions.

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Table 1. Summary of Hydraulic Conductivity and Storage Coefficient Data Available Prior to the June 1992 Aquifer Test.

Zone	Source	Well Number or Location	Depth (ft)	Hydraulic Conductivity (ft/day)	Storage Coefficient (dimensionless)	
Upper	Slug Test (G&M, 1986b)	GM-1	36	3.07	0.01	
(Layer 1)	Slug Test (G&M, 1986b)	GM-2	44	0.25	0.1	
	Slug Test (G&M, 1986b)	GM-3	36	0.47	0.1	
	Aquifer Test (G&M, 1986b)	B-1	10.5	0.51	--	
	Aquifer Test (G&M, 1986b)	B-10	35.5	17.01	--	
	Aquifer Test (G&M, 1986b)	B-11	25.5	5.67	--	
	Aquifer Test (G&M, 1986b)	B-15	45.5	3.97	--	
				4.42	0.07	Avg.
Intermediate (Layer 2)	Aquifer Test (1986b)	WGK Plant	65	441.18	0.04	
Deep (Layer 3)	Aquifer Test (G&M, 1986b)	Mobil Oil Corp. St. Clair County TZN, R10W Section 25	114	387.70	0.1	
	Aquifer Test (G&M, 1986b)	Ranney Well Sauget Site R	99	374.33	0.082	
				381.02	0.09	Avg.

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Table 2. Estimates of aquifer characteristics obtained through interpretation of data from the June 1992 aquifer test, Sauget, Illinois.

Well Number	Method	T* (ft^2/d)	K** (ft/d)	S***
Intermediate Zone				
P-5	Theis	38,000	420	0.012
P-9	Theis	22,000	240	0.013
P-10	Theis	22,000	240	0.0083
B-24C	Theis	34,000	380	0.0042
	Cooper-Jacob	32,000	360	0.0045
B-26B	Theis	32,000	360	0.0065
	Neuman	29,000	320	0.007
Deep Zone				
GM-57C	Hantush	31,000	340	0.0004
	Neuman	30,000	330	0.0055
GM-56C	Hantush	16,000	170	0.0013
	Neuman	24,000	270	0.016
GM-28C	Hantush	31,000	350	0.0001
	Average:	28,400	315	0.007

* Transmissivity (for the combined Intermediate/Deep Zones)

** Hydraulic Conductivity

*** Storage Coefficient (Specific Yield for Neuman Method)

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Table 3. Summary of Hydraulic Parameters used in the Monsanto Model.

Parameter Type	Zone	Value	Representation
Hydraulic Conductivity			
K(ft/day)	1	6.2	Water-table Layer (Cahokia Alluvium)
K(ft/day)	4	1.0	Site R (Water-Table Layer)
K(ft/day)	5	0.4	Mississippi River (Water-Table Layer)
Transmissivity			
$T(ft^2/day)$	2	15,000	Intermediate Layer (Henry Formation)
$T(ft^2/day)$	3	15,000	Deep Layer (Henry Formation)
Vertical Leakance			
$Kv(day^{-1})$	1	0.0063	Water-Table Layer/Intermediate Layer
$Kv(day^{-1})$	2	1.00	Intermediate Layer/Deep Layer
$Kv(day^{-1})$	3	$1.0x10^{-9}$	Water-table/Intermediate Layer in the landfill area
$Kv(day^{-1})$	4	0.42	Mississippi River/Intermediate Layer
$Kv(day^{-1})$	5	0.23	Intermediate Layer/Deep Layer (Mississippi River)
Recharge			
R(ft/day)	1	0.002	Water-Table Layer
R(ft/day)	2	0.084	Mounding at the W.G. Krummrich Plant (Water-Table Layer
R(ft/day)	3	0.0006	Landfill Cap at Site R (Water-Table Layer)

MONSANTO\MODEL RPT\TABLE.ANY642.11

Table 4. Comparison between observed and computed water levels in the steady-state calibration.

Well	Row	Column	Layer	Observed Head (ft above msl)	Computed Head (ft above msl)	Residual (ft)
GM-1	26	32	1	393.970	395.077	-1.11
GM-4A	33	22	1	388.720	389.829	-1.11
GM-4B	33	22	2	388.510	389.493	-0.983
GM-4C	33	22	3	390.530	389.492	1.04
GM-5	36	20	1	387.930	388.705	-0.775
GM-6A	38	24	1	389.390	390.439	-1.05
GM-6B	38	24	2	389.440	390.124	-0.684
GM-9A	35	29	1	394.810	392.909	1.90
GM-9B	35	29	2	391.540	392.427	-0.887
GM-9C	35	29	3	391.250	392.426	-1.18
GM-10A	28	28	1	395.740	396.468	-0.728
GM-10B	28	28	2	391.520	392.023	-0.503
GM-10C	28	28	3	391.400	392.009	-0.609
GM-11	24	30	1	392.530	392.929	-0.399
GM-12A	30	32	1	393.270	394.804	-1.53
GM-12B	30	32	2	393.600	394.465	-0.865
GM-12C	30	32	3	393.260	394.464	-1.20
GM-15	29	30	1	392.960	393.918	-0.958
GM-16A	25	27	1	391.480	391.800	-0.320
GM-16B	25	27	2	391.420	391.468	-0.048
GM-17A	36	23	1	389.220	390.255	-1.03
GM-17B	36	23	2	389.220	389.936	-0.716
GM-17C	36	23	3	388.950	389.935	-0.985
GM-18A	42	21	1	388.530	389.171	-0.641
GM-18B	42	21	2	388.610	388.855	-0.245
GM-19B	41	17	2	386.530	386.498	0.03192
GM-19C	41	17	3	386.580	386.497	0.08292
GM-20A	38	18	1	387.260	387.388	-0.128
GM-20B	38	18	2	387.110	387.072	0.03819
GM-22A	40	16	1	386.710	386.691	0.01897
GM-26A	31	16	1	386.380	386.443	-0.063
GM-26B	31	16	2	386.110	386.127	-0.017
GM-27C	36	8	3	383.040	382.213	0.827
GM-28B	42	10	3	382.700	382.531	0.169
GM-29	32	28	1	397.480	397.467	0.01282
GM-30	32	31	1	395.380	394.289	1.09
GM-31C	44	22	3	388.780	389.080	-0.300

Table 4. Comparison between observed and computed water levels in the steady-state calibration. (continued)

Well	Row	Column	Layer	Observed Head (ft above msl)	Computed Head (ft above msl)	Residual (ft)
GM-45	30	28	1	403.040	402.708	0.332
GM-46	31	29	1	401.520	401.128	0.392
GM-54B	44	20	2	388.390	388.417	-0.027
GM-55C	44	9	3	383.260	382.366	0.894
GM-56C	40	9	3	383.490	382.359	1.13
GM-57C	38	9	3	382.910	382.208	0.702
GM-58A	44	21	1	388.540	389.212	-0.672
GM-60A	36	15	1	386.010	385.941	0.06854
GM-60B	36	15	2	386.020	385.612	0.408
GM-60C	36	15	3	386.080	385.611	0.469
GM-61A	43	17	1	386.810	387.066	-0.256
GM-62A	41	13	1	385.420	385.365	0.05530
GM-62B	41	13	2	385.420	384.728	0.692
GM-62C	41	13	3	385.440	384.726	0.714
GM-63A	42	15	1	386.170	386.155	0.01485
B-21B	37	11	2	384.250	383.640	0.610
B-22A	37	9	1	396.300	392.283	4.02
B-24A	39	10	1	398.110	395.667	2.44
B-24B	39	10	2	383.170	382.659	0.511
B-24C	39	10	3	383.270	382.668	0.602
B-26A	41	10	1	392.500	394.400	-1.90
B-26B	41	10	2	383.200	382.645	0.555
B-27B	44	11	2	383.800	383.171	0.629
B-28A	43	10	1	394.320	393.846	0.474
B-28B	43	10	2	383.280	382.774	0.506
B-29A	43	12	1	394.090	390.691	3.40
B-31C	37	12	3	384.880	384.315	0.565
P-1	44	9	2	383.000	382.342	0.658
P-8	40	9	2	382.730	382.317	0.413
P-12	37	8	2	382.530	382.194	0.336
BK-3	30	29	2	392.130	392.614	-0.484
WB-6	39	33	1	394.240	396.027	-1.79

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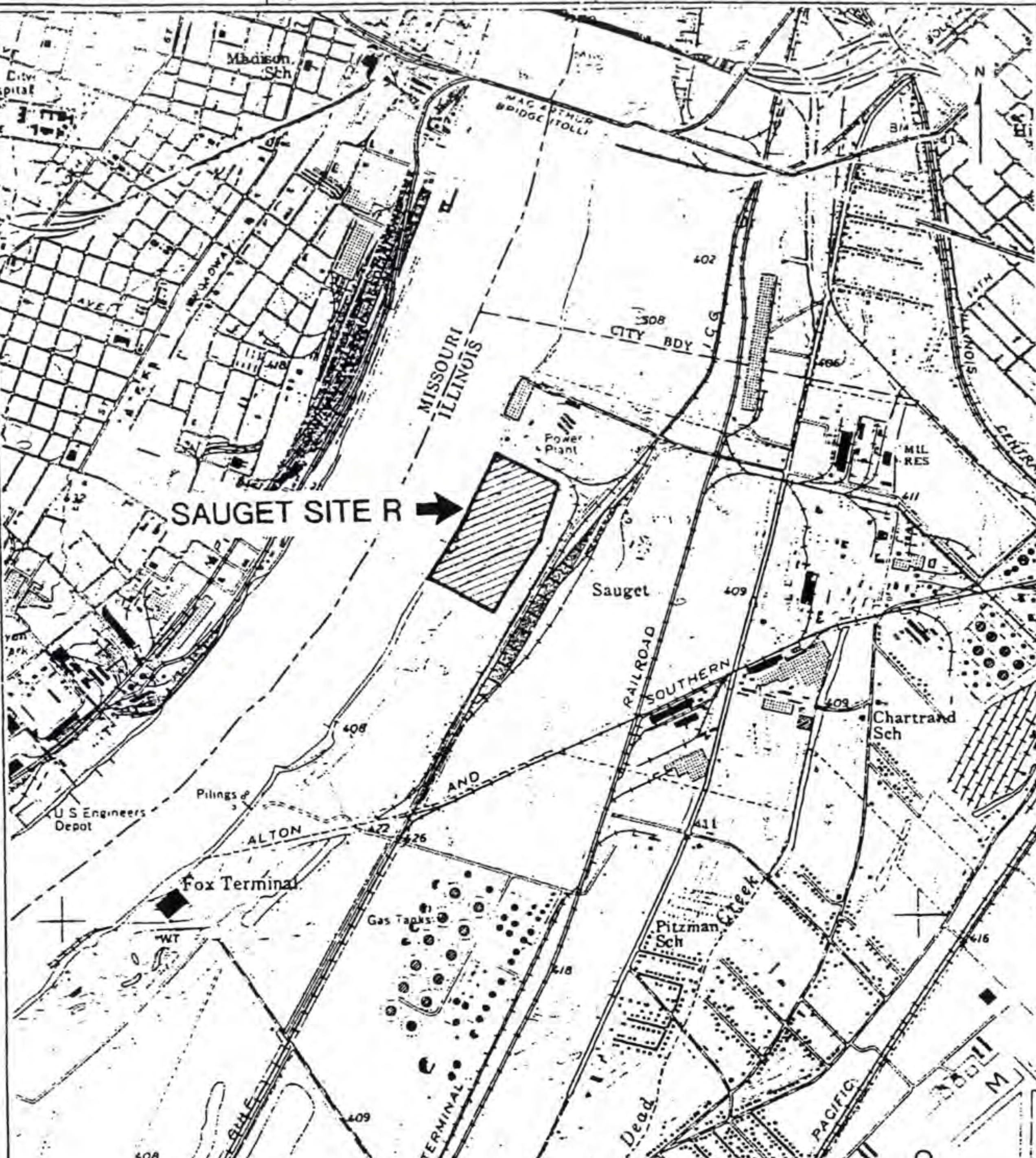

SOURCE: USGS CAHOKIA QUAD, 1974

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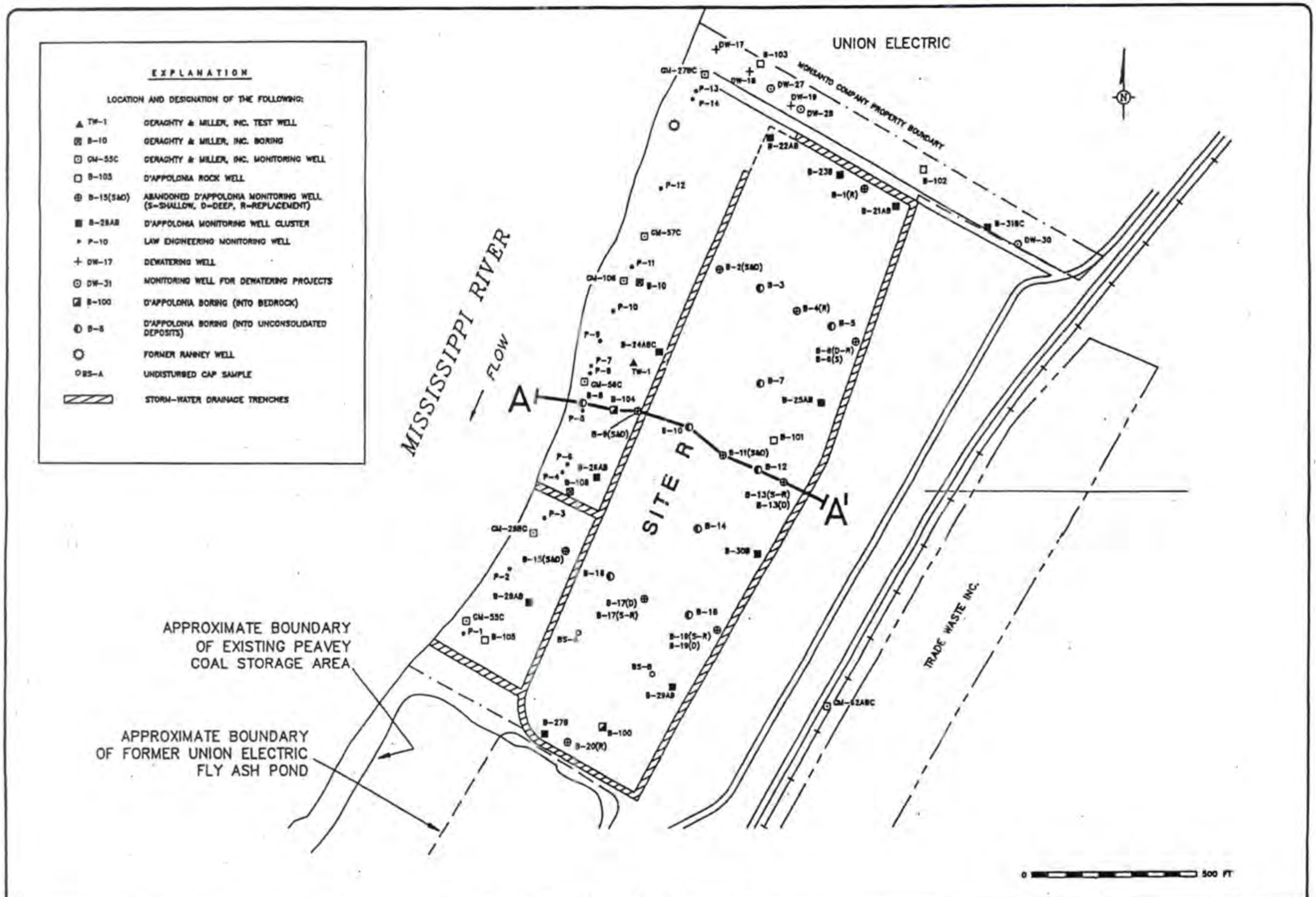
SUBJECT: LOCATION OF SAUGET SITE R, Monsanto Company, Sauget, Illinois

FIGURE 1



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LOCATIONS OF MONITORING WELLS AND SOIL BORINGS, SAUGET SITE R

MONSANTO COMPANY
SAUGET, ILLINOIS

FIGURE 2

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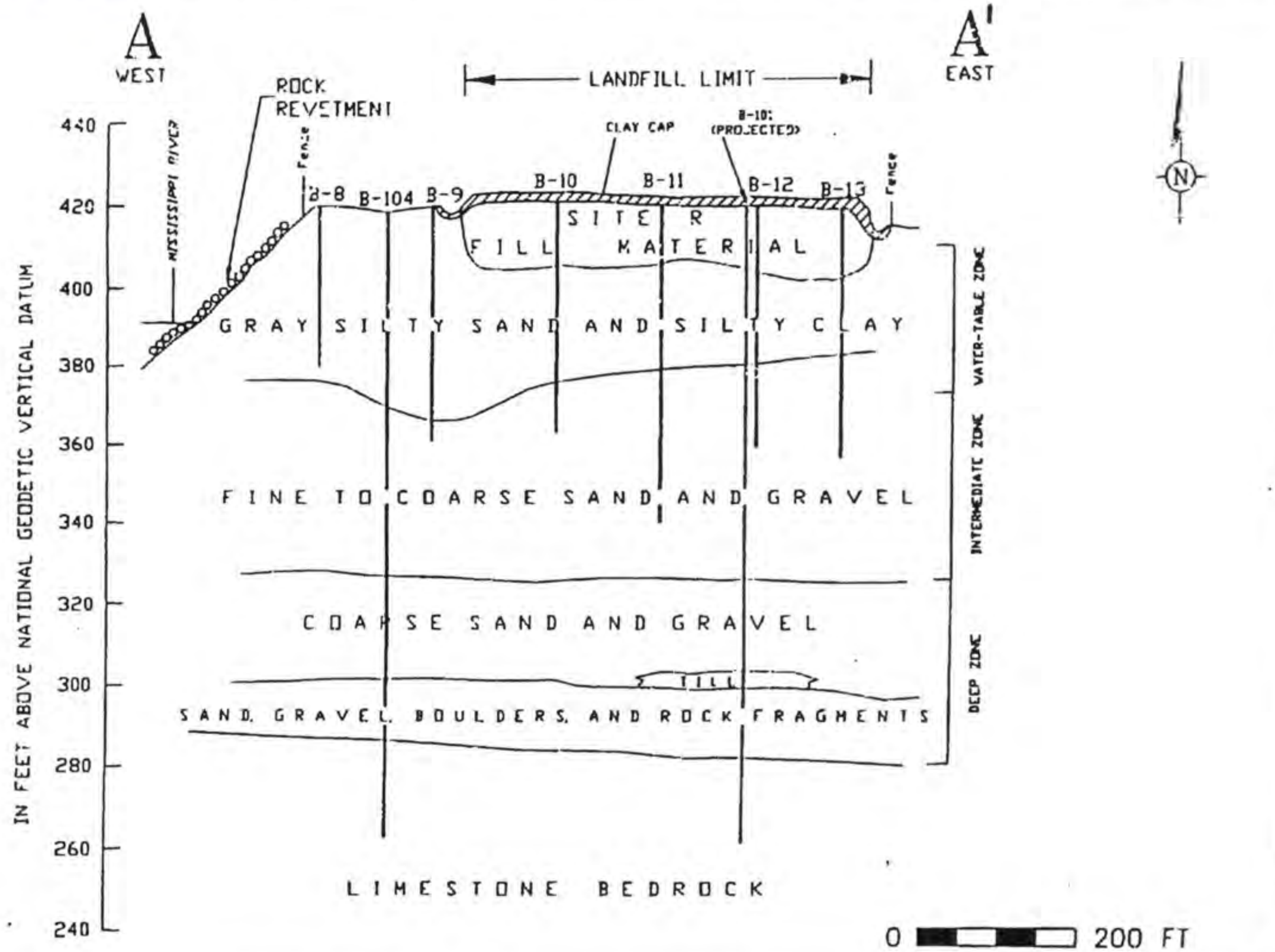

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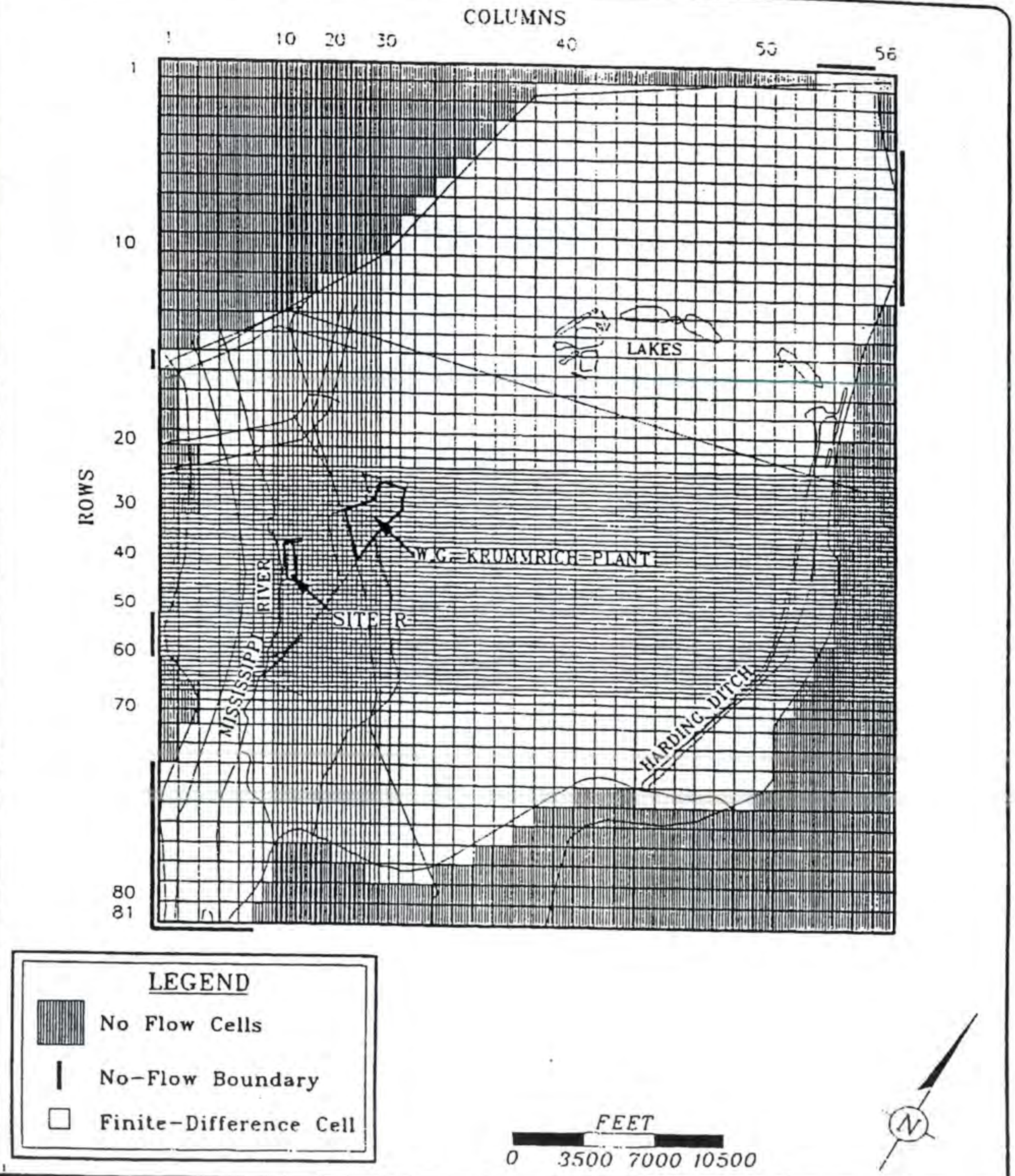
CROSS SECTION A-A', SAUGET SITE R, Monsanto Company, Sauget, Illinois

FIGURE 3



DRAFTER: CJS | APPROVED: JOR | CHECKED: JOR | DRAWING: MONSANTO | FILE NO.: FIG4.DWG | PRJCT NO.: NY642.08 | DWG DATE: 18NOV92

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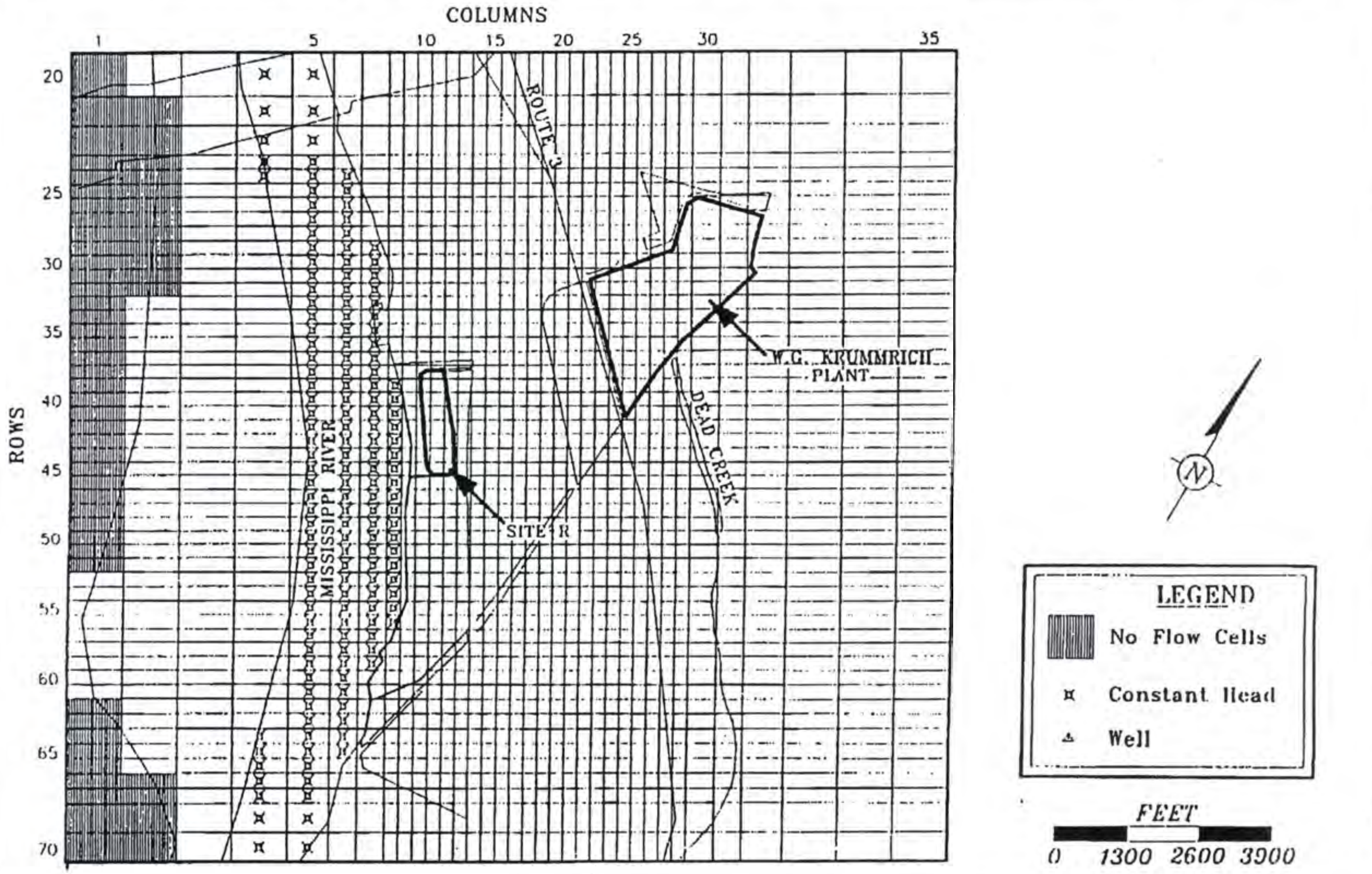
Finite-Difference Mesh of the Monsanto Ground-Water Flow Model.

Monsanto Company
Sauget, Illinois

FIGURE 4

DWG DATE: 18NOV92 | PRJCT NO.: NY642.08 | FILE NO: FIG5.DWG | DRAWING: MONSANTO | CHECKED: JOR | APPROVED: JOR | DRAFTER: CMS



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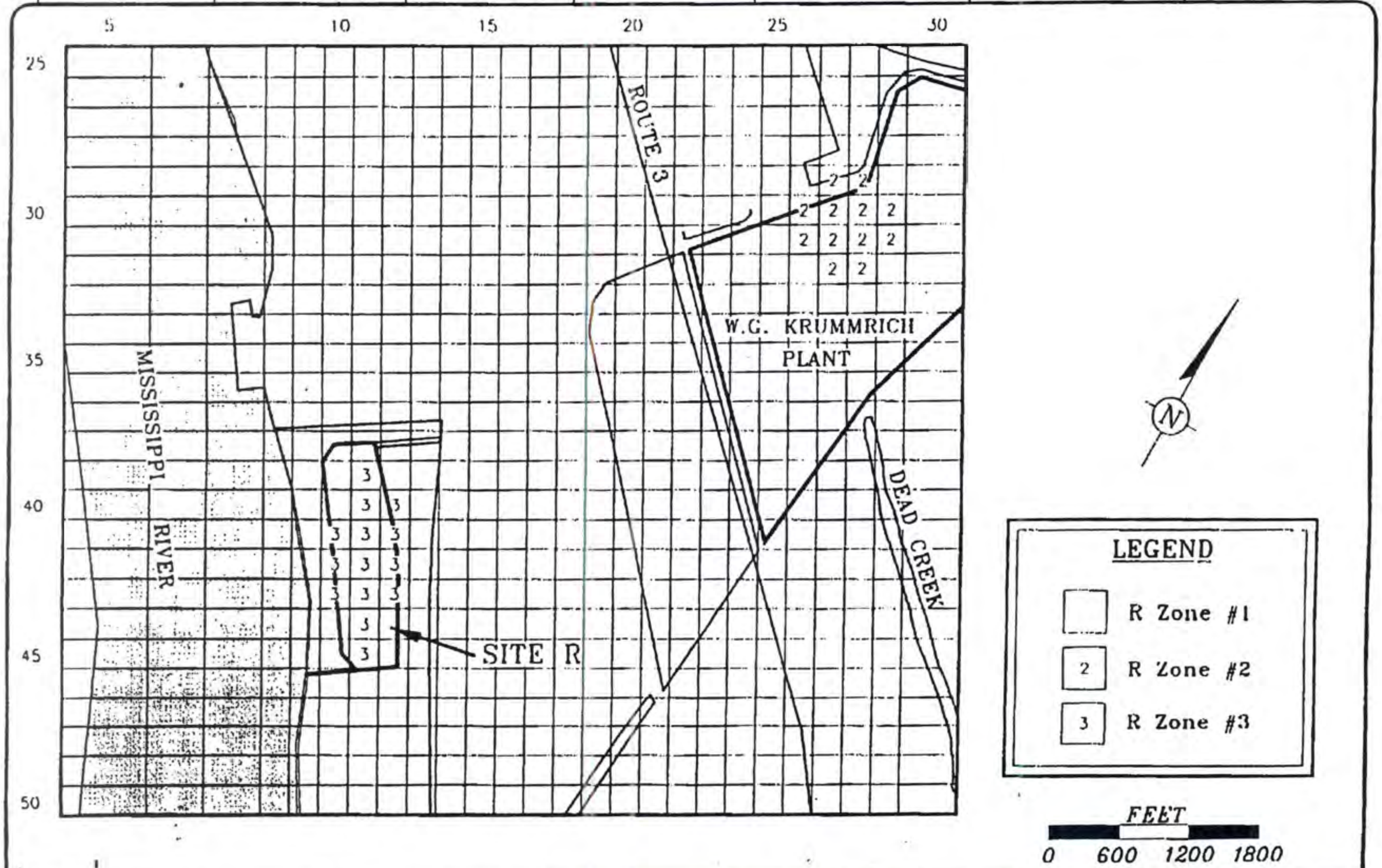
Portion of the Finite-Difference Mesh near Sauget Site R Illustrating Placement of Boundary Conditions in Layer 1.

Monsanto Company
Sauget, Illinois

FIGURE 5

DWG	18NOV92	PRJCT NO: NY642.08	FILE NO.: FIG6.DWG	DRAWING: MONS...[illegible]	CHECKED: JOR	APPROVED: JOR	DRAFTER: CMS



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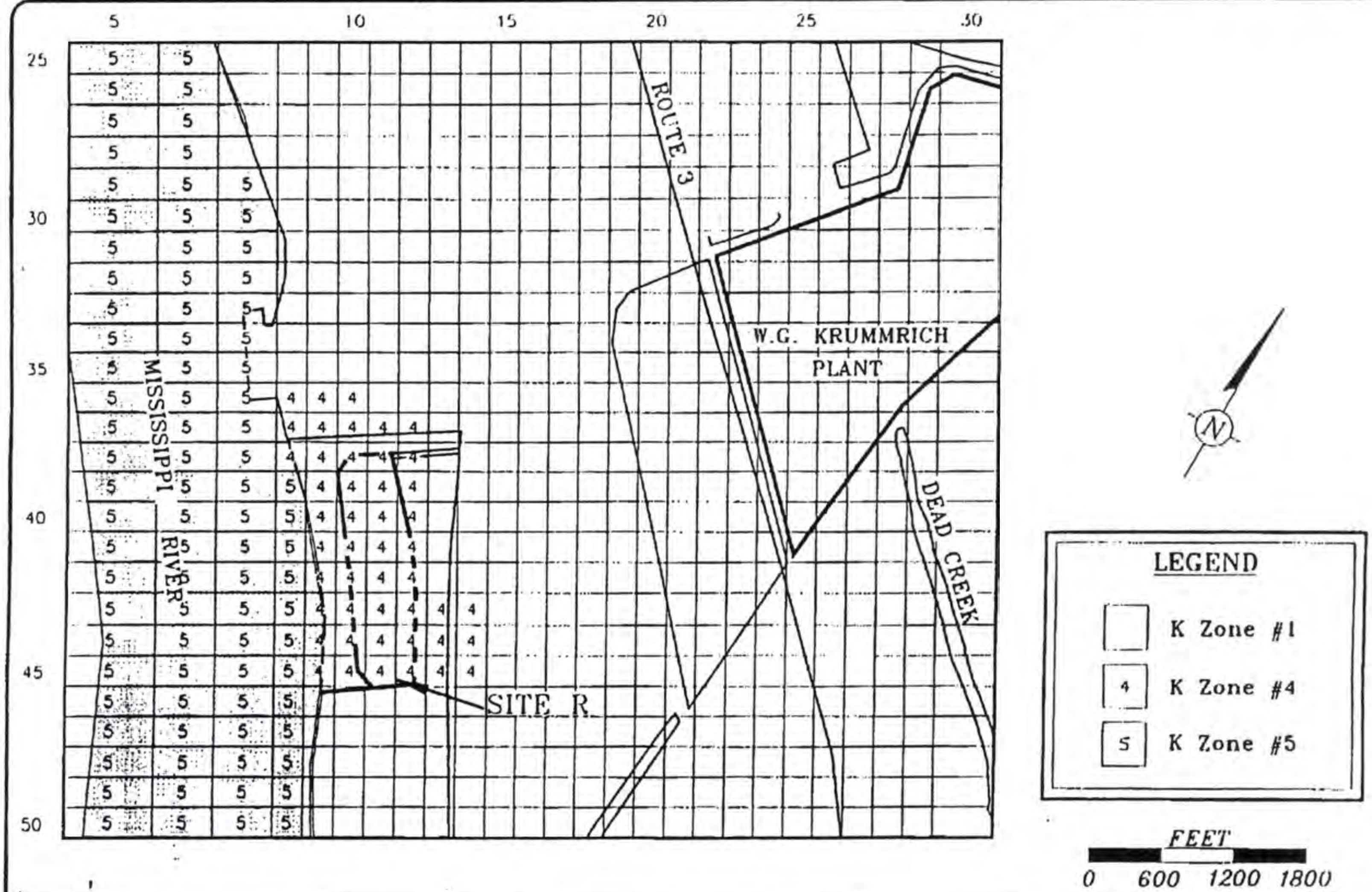
Portion of the Finite-Difference Mesh near Sauget Site R Illustrating Placement of Recharge Zones in Layer 1.

Monsanto Company
Sauget, Illinois

FIGURE 6

DWG DATE: 18NOV92	PRJCT NO.: NY642.08	FILE NO.: FIG7.DWG	DRAWING: MONSANTO	CHECKED: JOR	APPROVED: JOR	DRAFTER: CMS



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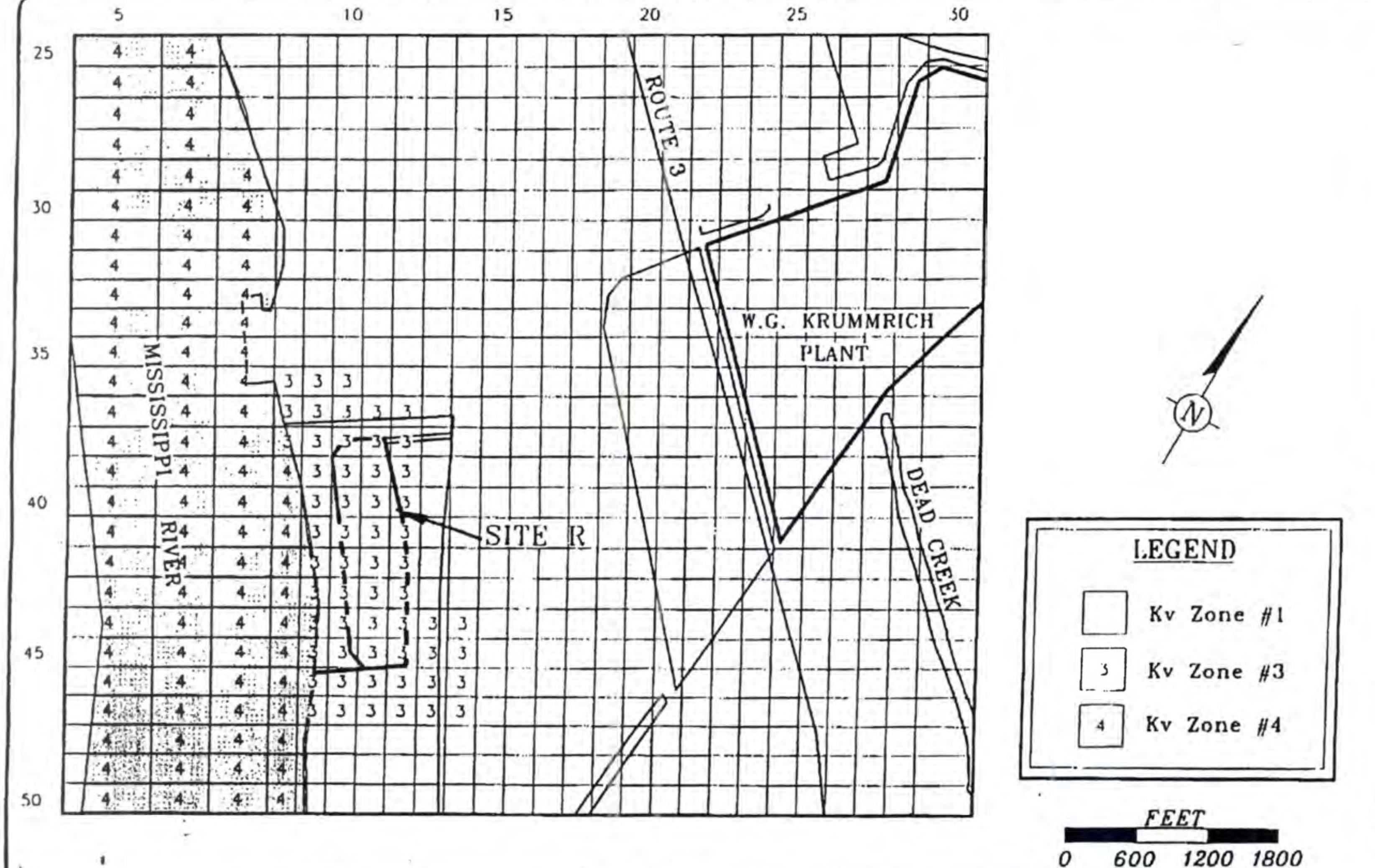
Portion of the Finite-Difference Mesh near Sauget Site R Illustrating Placement of Hydraulic Conductivity Zones in Layer 1.

Monsanto Company
Sauget, Illinois

FIGURE 7

DWG DATE: 18NOV92 | PRJCT NO.: NY642.08 | FILE NO.: FIG8.DWG | DRAWING: MONSANTO | CHECKED: JOR | APPROVED: JOR | DRAFTER: CMS



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Portion of the Finite-Difference Mesh near Sauget Site R Illustrating Placement of Leakance Zones in Layer 1.

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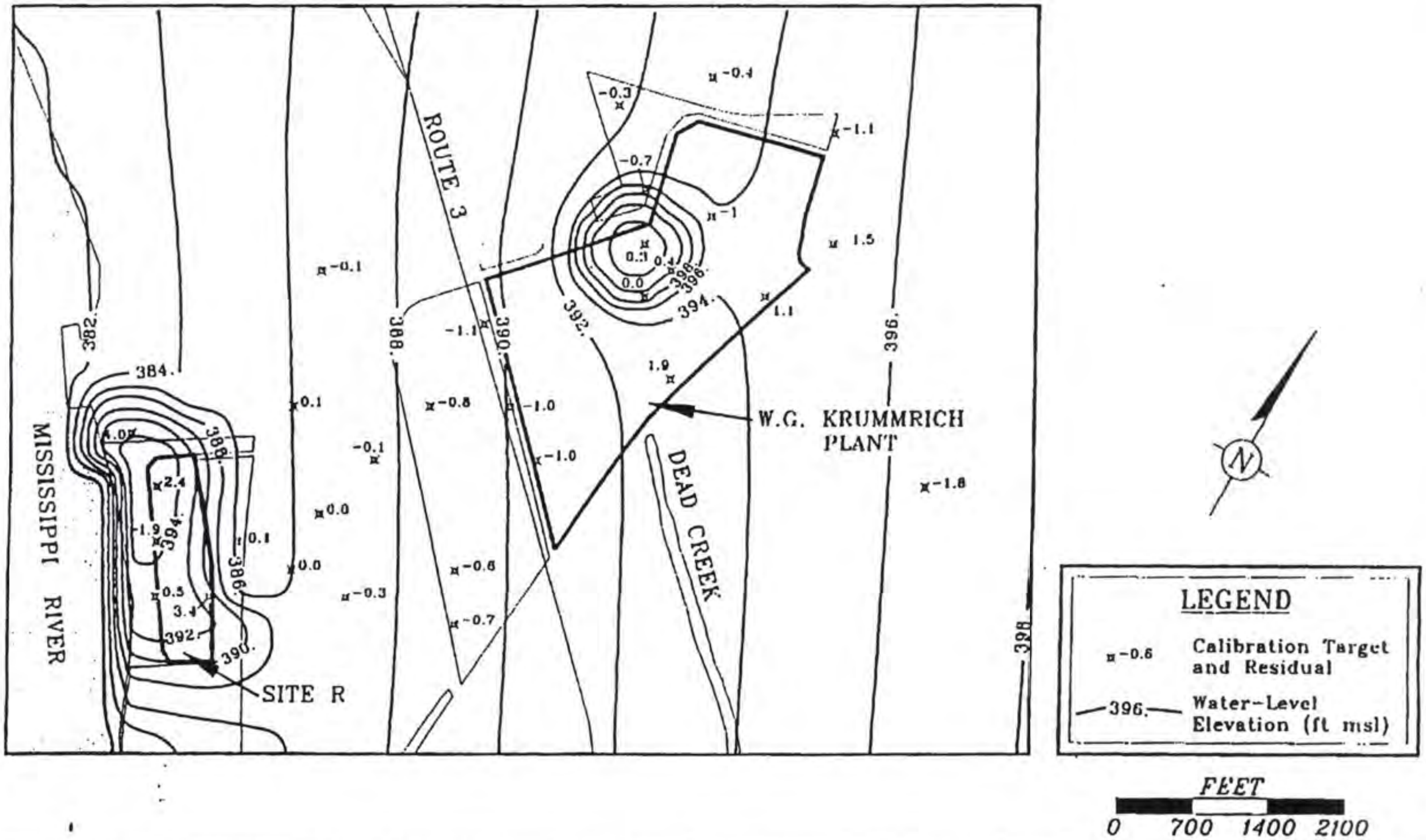
Sauget, Illinois

FIGURE 8

DWG DATE: 19NOV92 | PROJECT NO: NY642.08 | FILE NO: FIG9.DWG | DRAWING: MONSANTO | CHECKED: JOR | APPROVED: JOR | DRAFTER: CMS



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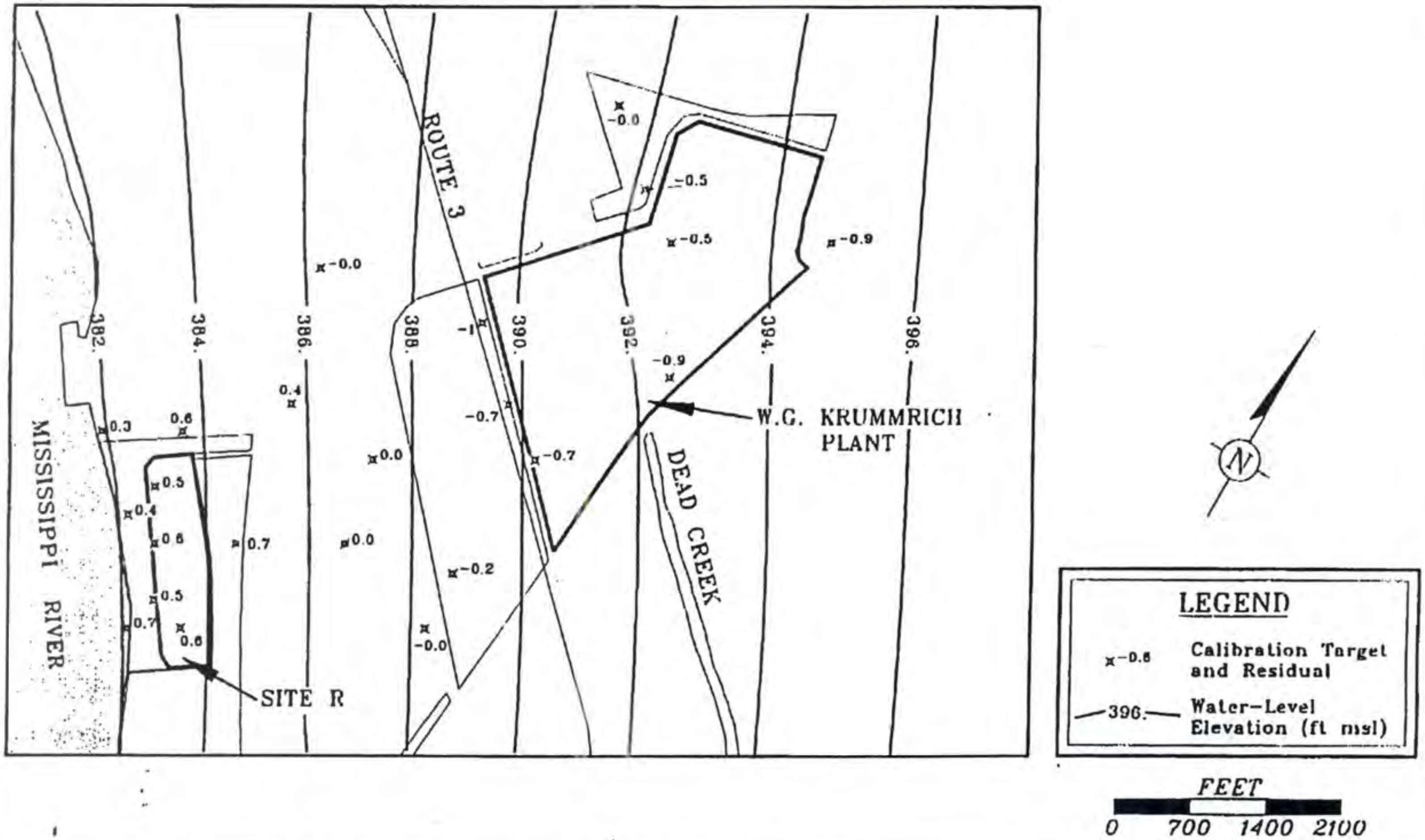
Environmental Services

Calibrated Water-Table Contours in Layer 1 Showing Residuals for the Steady-State Calibration.

Monsanto Company
Sauget, Illinois

FIGURE 9

DWG DATE: 19NOV92 | PRJCT NO.: NY642.08 | FILE NO.: FIG10.DWG | DRAWING: MONSANTO | CHECKED: JOR | APPROVED: JOR | DRAFTER: CMS



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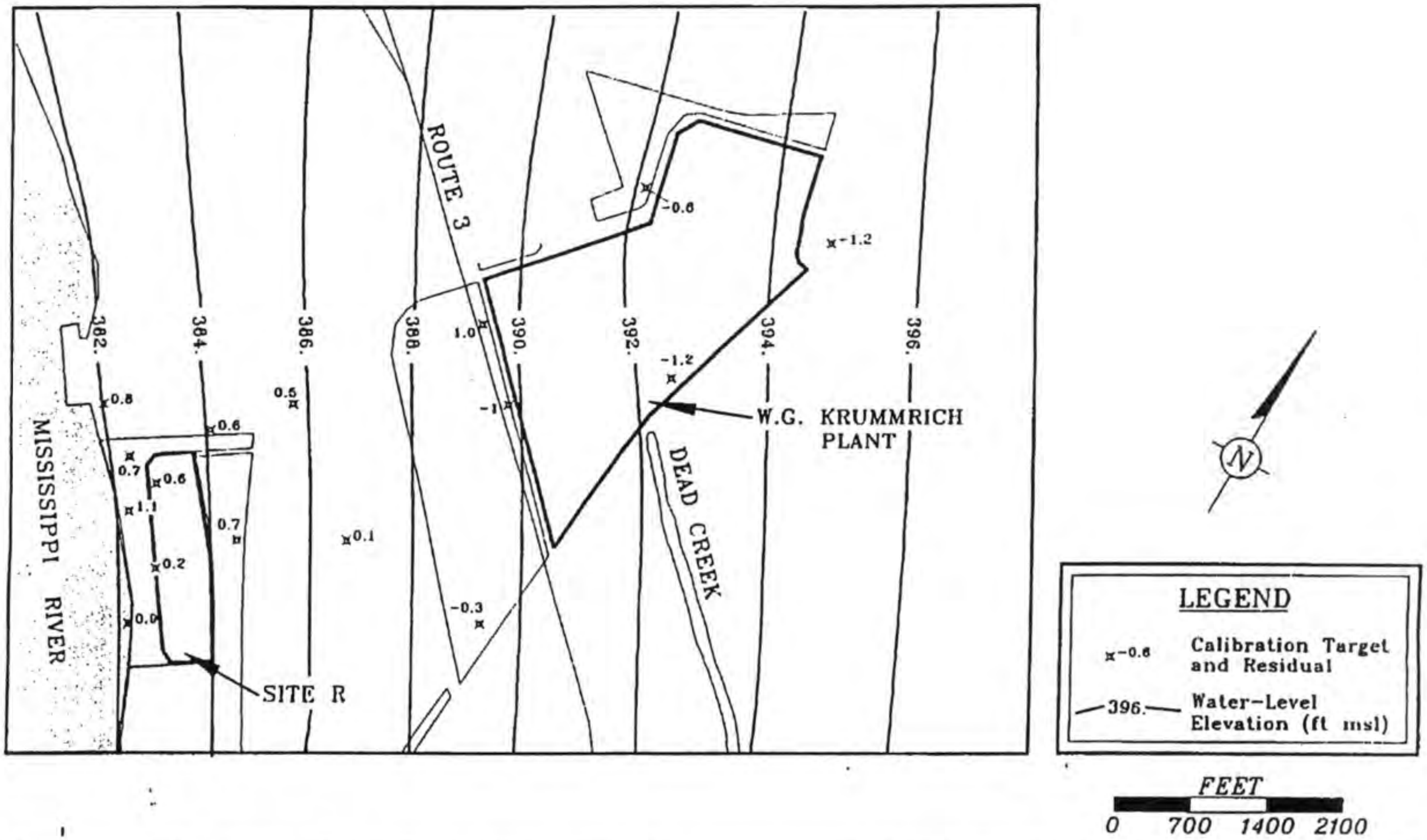
Environmental Services

Calibrated Potentiometric Surface in Layer 2 Showing Residuals for the Steady-State Calibration.

Monsanto Company
Sauget, Illinois

FIGURE 10

DWG D. _ 19NOV92 | PRJCT NO · NY642.08 | FILE NO: FIG11 DWG | DRAWING: MONSANTO | CHECKED: JOR | APPROVED: JOR | DRAFTER: CMS



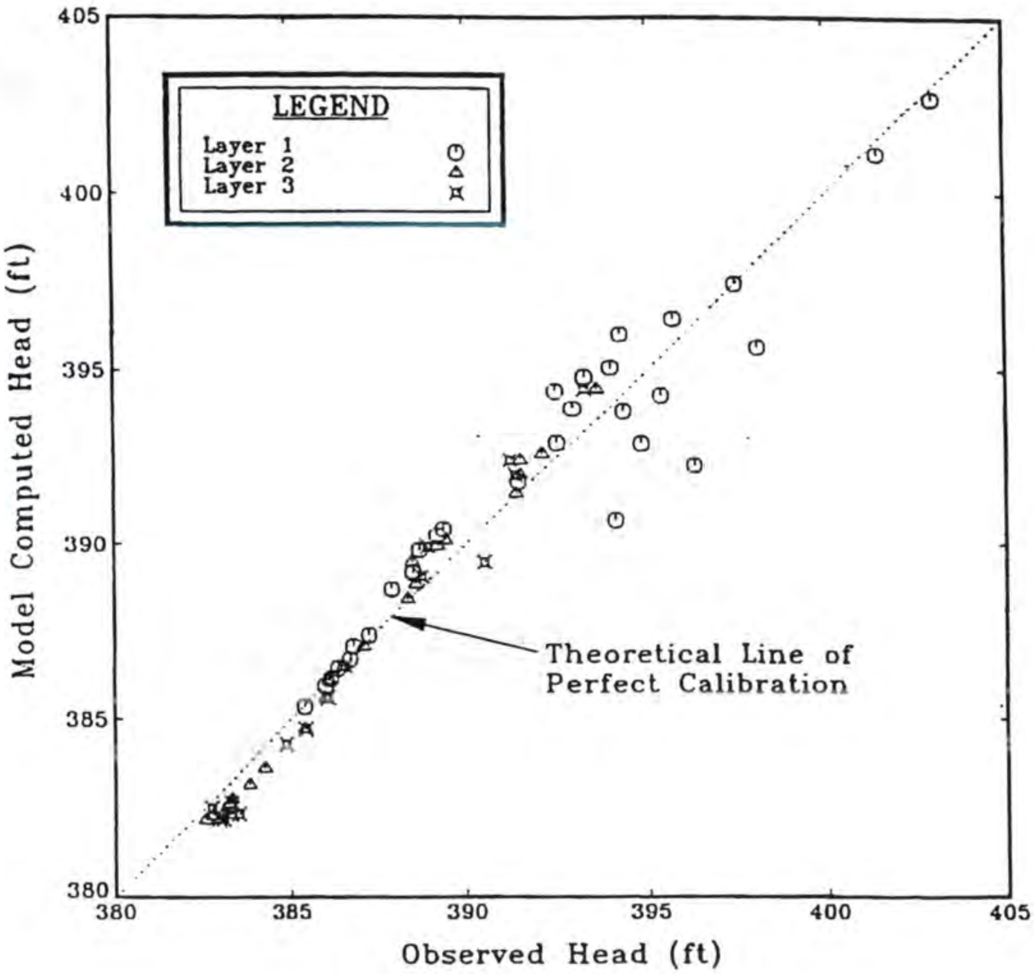
GERAGHTY & MILLER, INC.
Environmental Services

Calibrated Potentiometric Surface in Layer 3 Showing Residuals for the Steady-State Calibration.

Monsanto Company
Sauget, Illinois

FIGURE 11

DWG DATE: 19NOV92 | PRJCT NO.: NY642.08 | FILE NO.: FIG12.DWG | DRAWING: MONSANTO | CHECKED: JOR | APPROVED: JOR | DRAFTER: CMS



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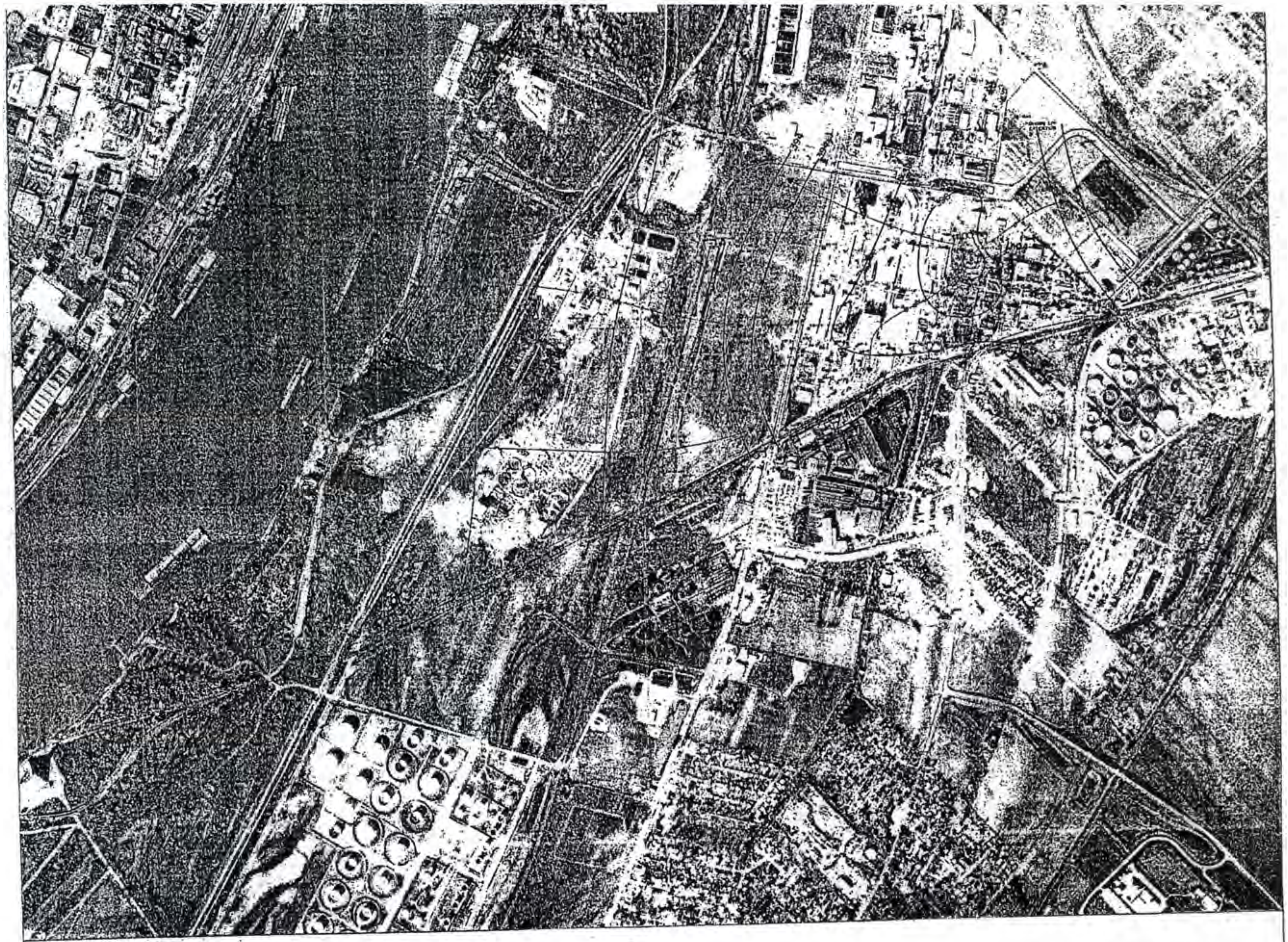
Comparison Between Observed and Computed Water Levels.

Monsanto Company
Sauget, Illinois

FIGURE 12



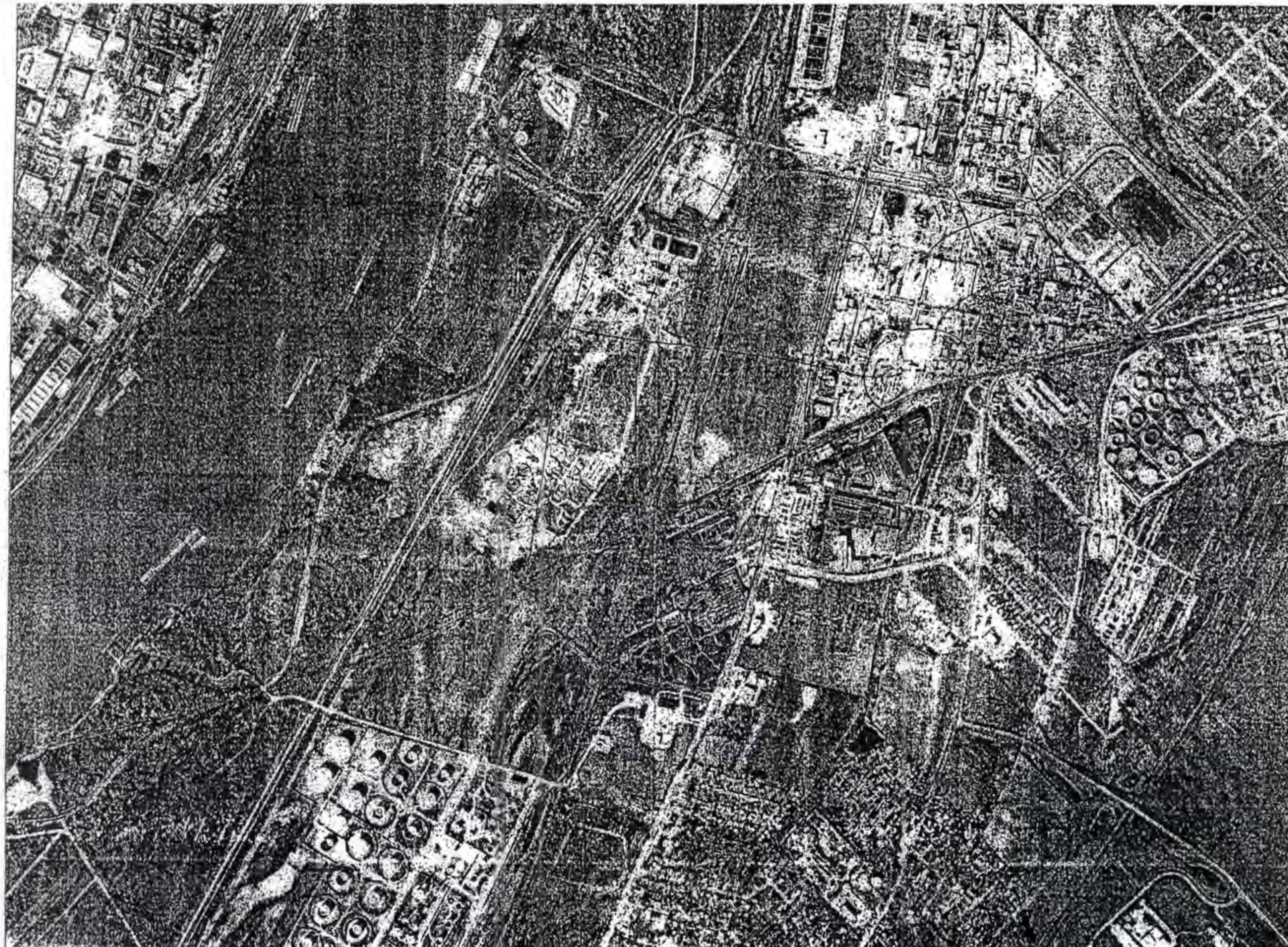
EXPLANATION

CONFIGURATION OF THE WATER TABLE

NOVEMBER 1985

MONSANTO COMPANY
Sauget, Illinois

FIGURE 13

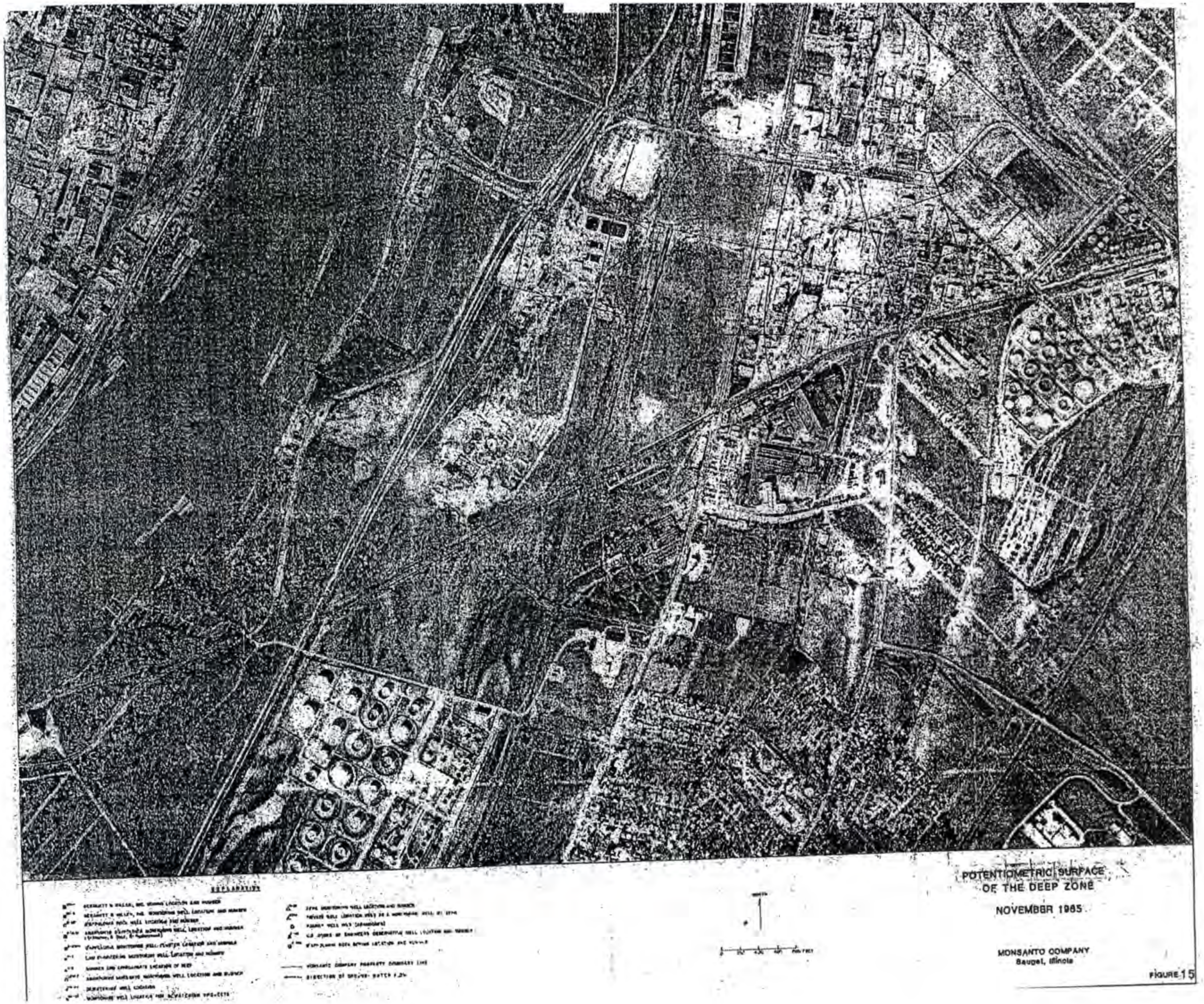


EXPLANATION

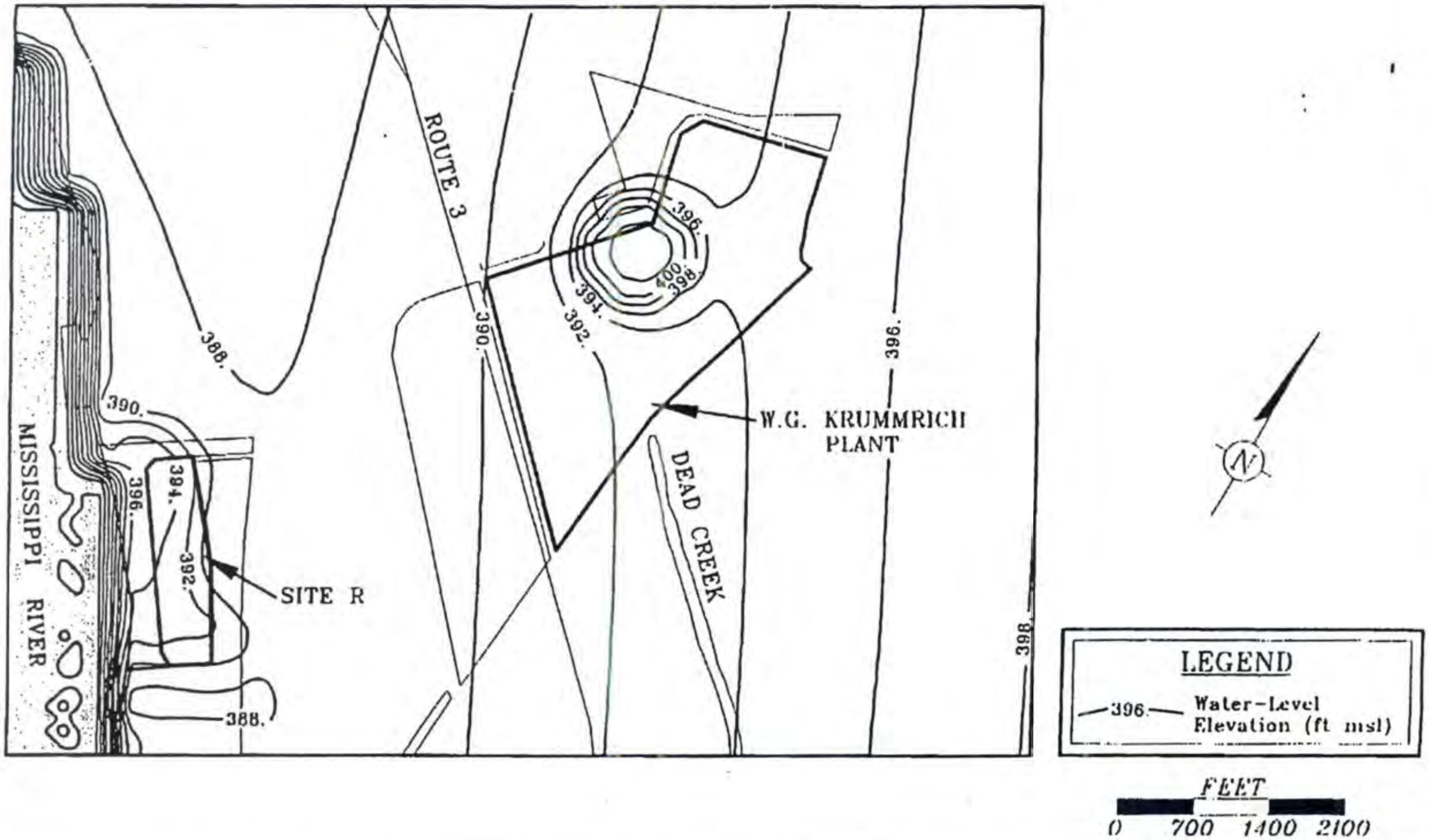
POTENTIOMETRIC SURFACE OF THE INTERMEDIATE ZONE NOVEMBER 1985

MONSANTO COMPANY
Sauget, Illinois

FIGURE 14


POTENTIOMETRIC SURFACE
OF THE DEEP ZONE
NOVEMBER 1985
MONSANTO COMPANY
Sauget, Illinois
FIGURE 15

DWG DATE: 19NOV92 | PRJCT NO.: NY642.08 | FILE N… DWG | DRAWING: MONSANTO | CHECKED: JOR | APPROVED: JOR | DRAFTER: CMS



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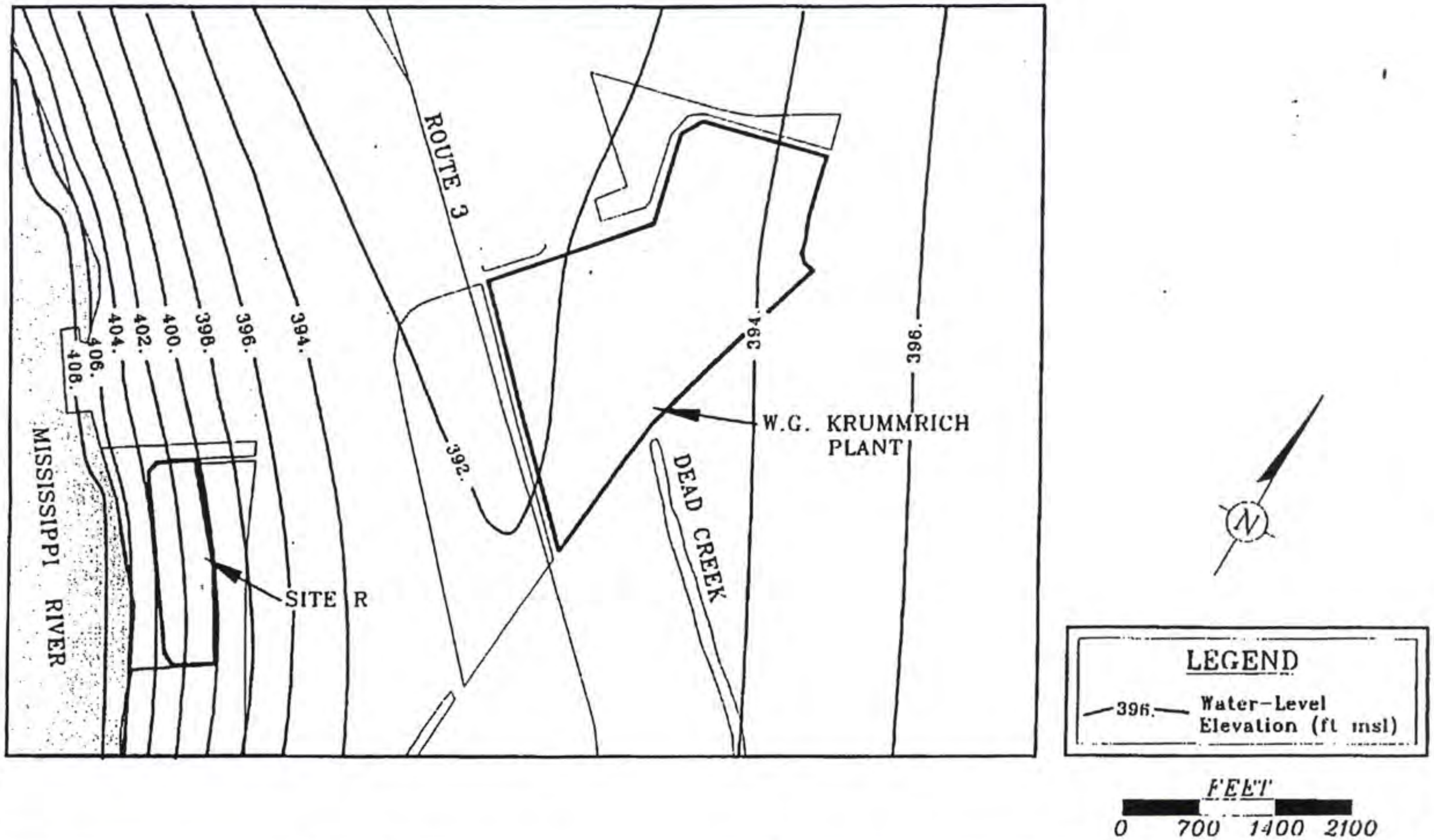
Model Calculated Water-Table Contours in Layer 1 for the Transient High-Water Analysis.

Monsanto Company
Sauget, Illinois

FIGURE 16

DWG DATE: 19NOV92 | PRJCT NO.: NY642.08 | FILE NO.: FIG17.DWG | DRAWING: MONSANTO | CHECKED: JOR | APPROVED: JOR | DRAFTER: CMS



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Model Calculated Potentiometric Surface in Layer 2 for the Transient High-Water Analysis.

Monsanto Company
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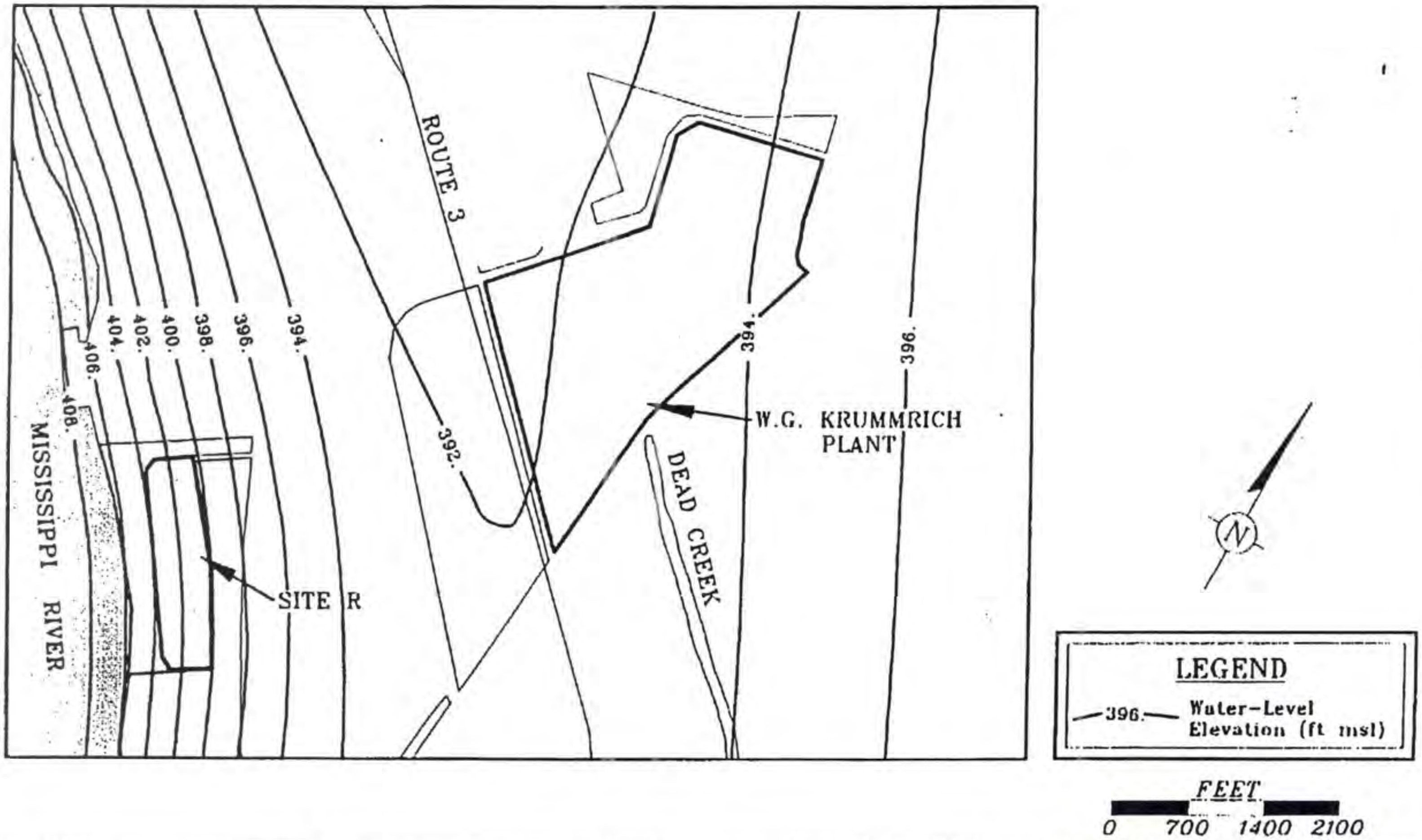
FIGURE 17

DWG DATE: 19NOV92	PRJCT NO.: NY642.08	FILE NO.: FIG18.DWG	DRAWING: MONSANTO	CHECKED: JGR	APPROVED: JGR	DRAFTER: CMS



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Model Calculated Potentiometric Surface in Layer 3 for the Transient High-Water Analysis.

Monsanto Company
Sauget, Illinois

FIGURE 18

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REPORT OF INVESTIGATIONS 191

GROUNDWATER GEOLOGY OF THE EAST ST. LOUIS AREA, ILLINOIS

BY

ROBERT E. BERGSTROM AND THEODORE R. WALKER

PRINTED BY AUTHORITY OF THE STATE OF ILLINOIS

URBANA, ILLINOIS

1956

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WILLIAM G. STRATTON, *Governor*
DEPARTMENT OF REGISTRATION AND EDUCATION
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ILLUSTRATIONS

GROUNDWATER GEOLOGY OF THE EAST ST. LOUIS AREA, ILLINOIS

BY

ROBERT E. BERGSTROM AND THEODORE R. WALKER

ABSTRACT

Geologic conditions favorable for large supplies of groundwater are among the factors promoting the concentrated industrial development of the Mississippi River bottomlands of the East St. Louis area, commonly known as the American Bottoms. The water-yielding deposits of the area are permeable sand and gravel in unconsolidated valley fill. The valley fill, which ranges to over 170 feet in thickness, consists partly of Recent alluvium and partly of older alluvium, some of which is glacial outwash material from the Upper Mississippi Valley. Valley-train sand and gravel occur beneath Recent alluvium in the northern part of the area and are present at the surface in terraces bordering the flood plain in the vicinity of Roxana. The lower alluvium south of the Missouri River mouth is older Missouri River sediment mixed with coarse glacial outwash material from the Upper Mississippi Valley. Although Recent cut-and-fill in this portion of the area has produced heterogeneity in the upper two-thirds of the valley fill, there is a general coarsening of material with depth. The most favorable water-yielding deposits usually occur below a depth of 60 to 90 feet, but clean sand and gravel are not present at all places on the American Bottoms. Distribution of permeable deposits and thickness of valley fill are controlled in part by the configuration of the bedrock valley floor.

Recharge of groundwater in the valley fill is by seepage from rainfall and floods and, in certain areas, by percolation from the Mississippi River and its tributaries. Geologic conditions appear favorable locally for greater groundwater exploitation, especially in some areas close to the river where permeable deposits are present and where river recharge might be induced by pumpage.

INTRODUCTION

Location

The East St. Louis area in southwestern Illinois includes the portions of Madison, St. Clair, and Monroe counties that lie within the valley bottom of the Missisippi River between Alton and Dupo, Ill. (fig. 1). The area is known locally as the *American Bottoms.* It includes about 175 square miles, is approximately 30 miles long, and has a maximum width of 11 miles. The principal cities are East St. Louis, Granite City, Wood River, and Alton.

The area has been mapped by the United States Geological Survey, and topographic maps of the following 7½-minute quadrangles are available: Alton, Bethalto, Columbia Bottom, Wood River, Granite City, Monks Mound, Cahokia, and French Village.

Purpose of Report

The East St. Louis area is one of the most highly industrialized areas in Illinois, and the demand for groundwater supplies

FIG. 1.—Index map showing location of East St. Louis area and major groundwater reports published since 1950 or in progress.

has been great. The total municipal and industrial pumpage of groundwater during 1951 averaged between 100 and 110 million gallons per day (Bruin and Smith, 1953, p. 5). The expansion of existing industries and the influx of new industries indicate that even greater demands will be made on groundwater reservoirs. To develop the groundwater resources to their full potential, careful consideration must be given to the geologic conditions that control the occurrence of groundwater in the area. This report summarizes these conditions and indicates areas favorable or unfavorable for the development of additional supplies. Emphasis is placed on geologic conditions controlling development of the large supplies needed for municipal and industrial purposes. Engineering aspects of the problem, involving detailed hydrologic and production data, have been under investigation for a number of years by the Illinois State Water Survey (Bruin and Smith, 1953).

ACKNOWLEDGMENTS

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We were assisted by many members of the Geological Survey staff, particularly those of the Division of Groundwater Geology and Geophysical Exploration. Many helpful suggestions and criticisms were made by M. M. Leighton, G. B. Maxey, J. C. Frye, F. C. Foley, H. B. Willman, Leland Horberg, and G. E. Ekblaw.

PREVIOUS INVESTIGATIONS

Early references to the geology of the East St. Louis area are contained in the reports on Madison and St. Clair counties of the first Geological Survey of Illinois, directed by A. H. Worthen (cited below). Subsequent reports on stratigraphy, physiography, and mineral resources were published by Fenneman, and a report on the groundwater resources was published by Bowman and Reeds. The Fenneman and Bowman reports, listed below, have been the primary sources of general geologic information on the area. Other geologic work in the vicinity of East St. Louis has been in connection with larger areal studies or on individual geologic problems. The following publications are concerned with geology of the area, with or without special reference to groundwater:

Bell, A. H., 1929, The Dupo oil field: Illinois Geol. Survey Ill. Pet. 17.

Bowman, Isaiah, and Reeds, C. A., 1907, Water resources of the East St. Louis district: Illinois Geol. Survey Bull. 5.

Drushel, J. A., 1908, Glacial drift under the St. Louis loess: Jour. Geol. v. 16, p. 493-498.

Ekblaw, G. E., and Workman, L. E., 1933, Subsurface geology in the East St. Louis region (abst.): Illinois Acad. Sci. Trans., v. 26, no. 3, p. 101.

Englemann, George, 1947, Carboniferous rocks of St. Louis and vicinity: Am. Jour. Sci., 2nd ser., v. 3, p. 119-120.

Fenneman, N. M., 1907, Stratigraphic work in the vicinity of East St. Louis: Illinois Geol. Survey Bull. 4, p. 213-217.

———, 1909, Physiography of the St. Louis area: Illinois Geol. Survey Bull. 12.

———, 1911, Geology and mineral resources of the St. Louis quadrangle, Mo.-Ill.: U. S. Geol. Survey Bull. 438.

Flint, R. F., 1941, Ozark segment of Mississippi River: Jour. Geol., v. 49, p. 626-640.

Horberg, Leland, 1950, Bedrock topography of Illinois: Illinois Geol. Survey Bull. 73.

Leverett, Frank, 1870, The Illinois glacial lobe: U. S. Geol. Survey Mon. 38, p. 64.

————, 1895, The preglacial valleys of the Mississippi and its tributaries: Jour. Geol., v. 3, p. 740-763.

————, 1921, Outline of the Pleiscene history of Mississippi Valley: Jour. Geol., v. 29, p. 615-626.

Robertson, P., 1937, Drift exposures in St. Louis and St. Louis County (abst.): Missouri Acad. Sci. Proc., v. 3, no. 4, p. 129.

————, 1938, Some problems of the middle Mississippi River during Pleistocene time: St. Louis Acad. Sci. Trans., v. 29, no. 6, p. 169-240.

————, 1940, Some Pleistocene terraces of the Mississipi River (abst.): Geol. Soc. Am. Bull., v. 51, no. 12, pt. 2, p. 2041.

Rubey, W. W., 1952, Geology and mineral resources of the Hardin and Brussels quadrangles (in Illinois): U. S. Geol. Survey Prof. Paper 218.

Wanless, H. R., 1933, Pennsylvanian rocks of Madison and St. Clair counties, Illinois: Illinois Acad. Sci. Trans., v. 26, no. 3, p. 105.

Worthen, A. H., 1866, Madison County: Geol. Survey of Illinois, v. 1, p. 249-263.

————, 1866, St. Clair County: Geol. Survey of Illinois, v. 1, p. 231-248.

Of the following publications, pertaining more specifically to engineering phases of groundwater work, the report by Bruin and Smith contains the most recent and complete information on the hydrology and water quality in the American Bottoms:

Brittain, D., 1875, On the well at the Insane Asylum, St. Louis Co., Missouri: Am. Jour. Sci., 3rd ser., v. 9, p. 61-62.

Broadhead, G. C., 1878, On the well at the Insane Asylum, St. Louis Co.: St. Louis Acad. Sci. Trans., v. 3, p. 216.

Bruin, Jack, and Smith, H. F., 1953, Preliminary investigation of groundwater resources in the American Bottoms: Illinois Water Survey Rept. Inv. 17.

Gleason, C. D., 1935, Underground waters in St. Louis County and City of St. Louis, Missouri: Missouri Div. of Geol. Survey and Water Resources, Bienn. Rept. of State Geologist, 1933-34, app. 5.

Prout, H. A., 1853, Belcher's artesian well in St. Louis: Am. Jour. Sci., 2nd ser., v. 15, p. 460-463.

St. Louis Chamber of Commerce, 1950, The industrial water resources of the St. Louis area.

Searcy, J. K., Baker, R. C., and Durum, W. H., 1952, Water resources of the St. Louis area, Missouri: U. S. Geol. Survey Circ. 216.

Shepard, E. M., 1907, Underground waters of Missouri—their geology and utilization: U. S. Geol. Survey Water Supply Paper 195.

Suter, Max, 1942, Groundwater studies in the East St. Louis district: Illinois Engineer, v. 18, no. 2.

EXTENT AND RELIABILITY OF SUBSURFACE DATA

This report is based on a study of about 700 logs of wells and borings, supplemented by studies of available samples. Most of the logs are of water wells or of test borings made prior to the construction of water wells. Many are logs of borings made by the U. S. Corps of Engineers in connection with levee construction. A few are logs of oil wells or oil test holes. Most of the borings do not extend through the unconsolidated sediments lying above bedrock. Many of the wells were drilled to what the drillers assumed to be bedrock, but it is likely that many of these borings end at large boulders several feet above the bedrock, for nearby wells record greater depths to bedrock. The Corps of Engineers recognizes the possibility that many of their borings end with the bit resting on a boulder lying above bedrock; they label such depths "bit refusal" rather than "bedrock." The term "bit refusal" is preferred to an unqualified designation as bedrock in those cases where the drilling does not actually continue into bedrock for at least a few feet.

In mapping the surface of the bedrock, we have considered as reliable only those wells that have penetrated the underlying rock. The only wells that satisfy this requirement are the oil wells and oil test holes, and these are few. The reliability of the remainder of the logs is open to some question, so a subjective factor was involved in construction of the bedrock surface contour map.

Logs of oil wells and oil test holes are of little value in giving information on the lithology of the unconsolidated material in the American Bottoms because they lack detail in the upper sections. For information on the lithology of the valley fill, reliance must be placed upon logs of shallow borings. Logs obtained from the Corps of Engineers are considered to be the most reliable. The borings from which these logs were made were supervised by field engineers experienced in collecting and recording such data, and the sampling intervals were closely spaced. In addition, many of these logs have been compiled after mechanical analyses were made of the samples. Logs obtained from water-well drillers are less reliable, as many lack detail. Where drillers attempted to classify the sediments into grain sizes, a large personal factor was involved. For example, the sediment in many samples is described as "building sand" or "quicksand"; in such cases much has been left for us to interpret.

Some information also has been obtained from excavations made for the construction of piers and abutments for bridges across the Mississippi River. These give reliable information on bedrock elevations but at best furnish only very generalized information on the nature of the unconsolidated sediments. To supplement the data available on depth to bedrock, a refraction seismograph study was made at locations where well information was lacking.

An attempt was made to obtain additional information on the stratigraphy of the unconsolidated sediments by the electrical earth resistivity method. Twenty-five resistivity stations were set up adjacent to wells or borings for which detailed logs were available and which thus could serve as controls. The results of this work are inconclusive. We decided that unknown factors were influencing the resistivity readings, and this phase of the investigation was halted.

TOPOGRAPHY AND DRAINAGE

The Missouri and Mississippi rivers come together in the northern part of the area, about 5 miles downstream from Alton. Upward from this junction within the area of study and for several miles upstream, these two rivers flow southeast in the same valley, bordered on each side by bluffs of Mississippian limestone (tables 1 and 2). Below this junction the Mississippi River flows south across the area. Through the middle of the area the river valley crosses the western edge of a lowland cut in easily eroded Pennsylvanian ("Coal Measures") rocks and attains its maximum width (approximately 11 miles). In the southwestern part of the area the river crosses the more resistant Mississippian limestone and its valley narrows to about 3½ miles in width. At present, only in the area above Alton is the Mississippi River eroding the valley walls on the Illinois side. It is cutting along the western bluffs throughout the remainder of the area.

Along the river channel, the flood plain ranges in average elevation from 415 feet in the vicinity of Alton to 405 feet in the vicinity of Dupo. In this distance of 30 miles, the river falls 16 feet, a gradient of about 6 inches per mile.

In relatively recent geologic time, the Mississippi River has changed its course frequently in the East St. Louis area, producing a complex variety of land forms and

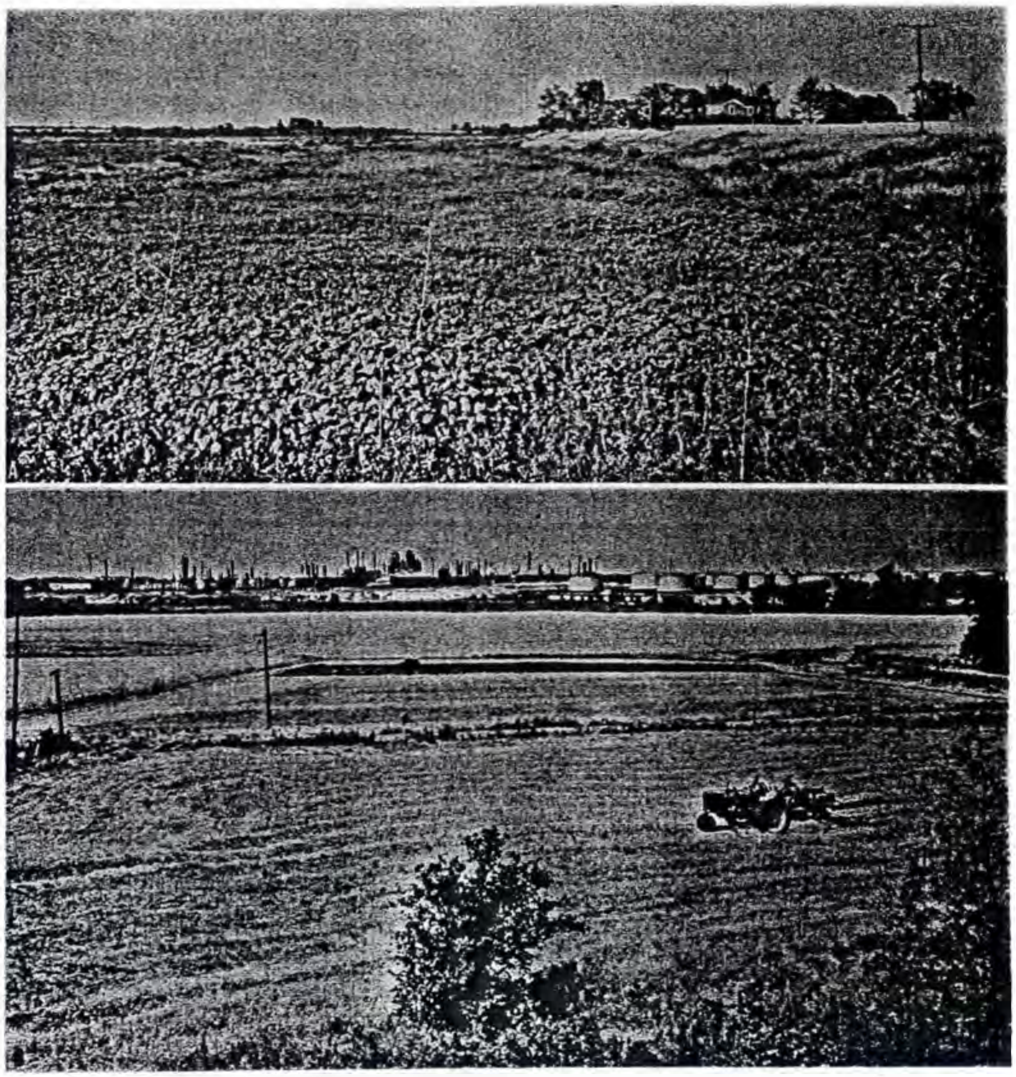

PLATE 1.—A. Horseshoe Lake bed, Madison County, Ill. View east from highway U.S. 66 near National City. B. Shell Oil Company Wood River Refinery, built on terrace above the Mississippi River flood plain. View southwest from bluffs east of Roxana, Madison County, Ill.

river deposits (Fenneman, 1909, p. 13, 29). Horseshoe Lake(pl. 1A) and other crescent-shaped lakes, swamps, and lowlands in the area mark the location of former meanders abandoned in the process of channel migration. The arcuate ridges and swales that border these meander loops on the concave side were formed as slackwater bars in former channels. East of the meander belt are discontinuous areas of poorly drained lowlands or backwater swamps which have been partially filled by silt and clay from floodwaters of the Mississippi and local tributaries.

In the northern part of the American Bottoms, deposits of sand and gravel occur in terraces that stand above the flood plain. They are eroded remnants of a valley fill of sand and gravel deposited by water from melting glaciers to the north, in the Mississippi drainage basin. These deposits formerly filled the valley to the present levels of the terraces. The low, broad ridge upon which East Alton, Wood River, Roxana,

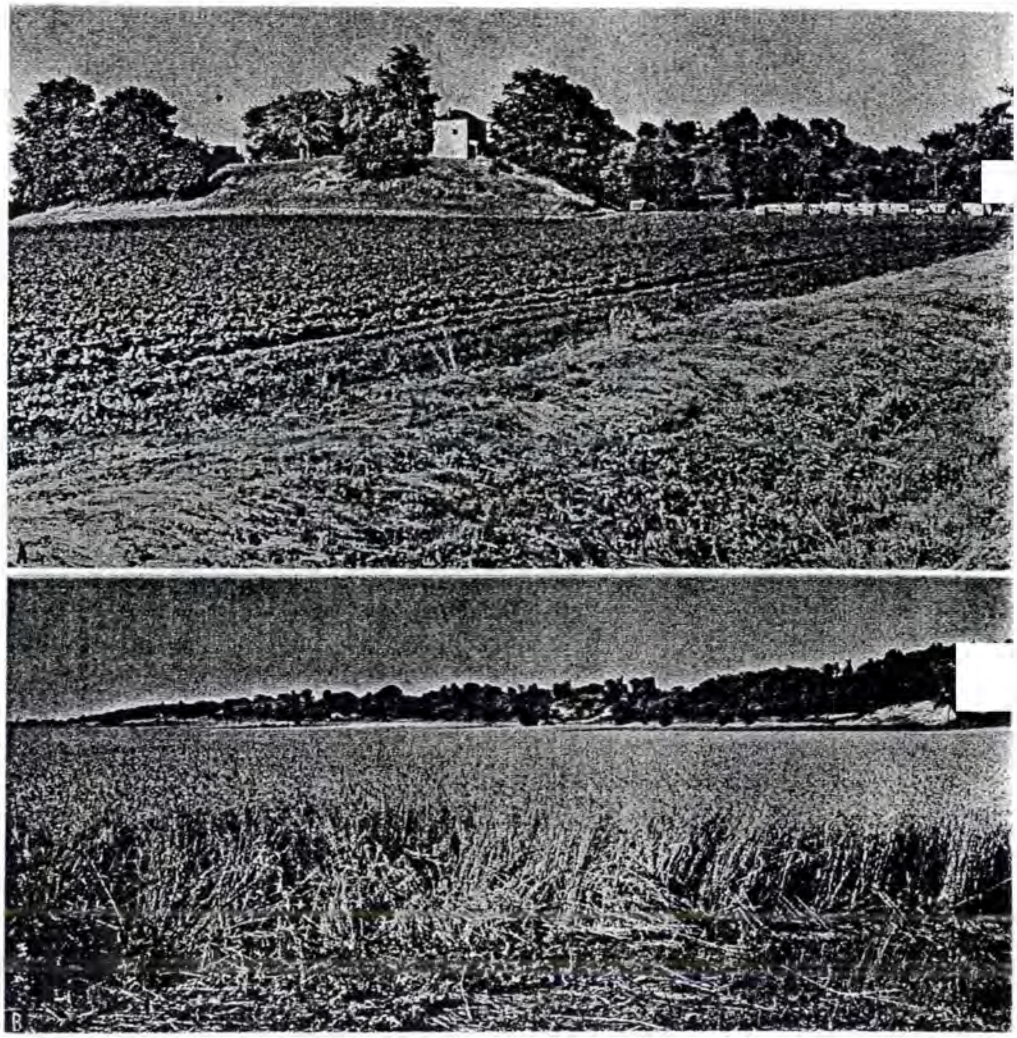

PLATE 2.—A. Mounds at Cahokia Mounds State Park, Madison Co. B. East bluffs bordering the American Bottoms. Looking northeast from 3 miles northeast of Horseshoe Lake, Madison Co.

and South Roxana are built is a terrace that stands 40 feet or more above the Mississippi River and 25 to 35 feet above the present flood plain (pl. 1B). The terrace is 440 to 450 feet above sea level. The front of the terrace has a sharp rise of 12 to 15 feet. This terrace level is also represented by low flat-topped knolls in the vicinity of Poag and just west of Indian Creek south of Roxana.

Many areas on the American Bottoms are somewhat above the flood plain but are below the level of the terrace at Wood River. North of Horseshoe Lake, the elevation of this intermediate level is 420 to 435 feet. It is more recent than the Roxana terrace but also may represent aggradation during late glacial time.

Between East St. Louis and the eastern bluff is a group of mounds occupying an area of 3 to 4 square miles (pl. 2A). The largest of these, Monks Mound, is about 85 feet high, whereas the smaller ones are only a few feet high. Although some of the mounds are symmetrical, steep, and cone-shaped, indicating an artificial origin, some of them may be remnants of an earlier higher flood plain.

The bluff that forms the eastern edge of the valley rises 150 to 200 feet above the valley bottom. Bedrock is well exposed in the bluffs on the Illinois side of the river in only two places, northwest of Alton in the northern part of the area and south of Stolle in the southern part. Most of the bluffs on the eastern side of the valley (pl. 2B) are covered by a mantle of glacial drift overlain by windblown silt called *loess*. With the exception of the two areas mentioned, the loess also blankets the face of the bedrock bluff. Between Edgemont and Caseyville, however, the loess cover is patchy and there are scattered outcrops of Pennsylvanian bedrock in the bluffs.

Many alluvial fans have been developed below the bluffs on the eastern side of the valley. These fans are composed predominantly of reworked loess which has been picked up by tributary streams in the upland and redeposited where the tributaries enter the main valley. As a result of the deposition of alluvial fans, the elevation of the valley bottom adjacent to the eastern bluff is 30 to 50 feet higher than the general elevation of the valley bottom. The alluvial fans, however, are not to be confused with the terraces of glacial sand and gravel mentioned above, for they gently slope and thin valleyward and have an entirely different lithologic composition.

The upland adjacent to the American Bottoms consists of broad, flat plains separated by relatively narrow, deep valleys. In most places the major tributary streams appear to follow preglacial bedrock valleys. The valley floors have relatively steep gradients as they join the main valley. In contrast, the Mississippi valley bottom slopes gently southward at an average rate of only about 6 inches per mile. In times of heavy rainfall the tributaries carry more water than normally can be confined within their banks in their lower courses across the Mississippi flood plain. Formerly this resulted in numerous floods along those portions of the tributaries that lie within the valley. As a corrective measure, the lower courses of the tributary streams have been straightened and levees constructed to prevent flooding of agricultural lands.

East of Dupo and south of Stolle, where easily dissolved Mississippian limestones are near the surface, the ground is pitted with hundreds of sinkholes 10 to 40 feet deep. This irregular sinkhole topography is markedly different from the flat divides and narrow valleys farther east.

OCCURRENCE AND MOVEMENT OF GROUNDWATER

General Principles

Water flowing over the ground or falling on the ground as rainfall seeps through openings between loose particles of the soil and percolates downward. Below a certain depth, all openings in the loose surface materials and underlying bedrock are filled with water.

The upper surface of the saturated zone is called the *water table*. Its position is determined by the depth at which water stands in wells, borings, and excavations. The water-table surface roughly parallels the surface topography, rising under the uplands and intersecting the ground surface along perennial streams, lakes, and swamps. Its position fluctuates from season to season and year to year. The water table is lowered during periods of prolonged drought; it rises during periods of excessive rainfall. In the East St. Louis region its position is normally at a depth of about 15 to 20 feet below the surface of the valley floor, although concentrated pumpage has lowered it considerably over much of the area.*

The water in the upper part of the saturated zone is unconfined and moves under the influence of gravity in the direction of the water-table slope. In wells that penetrate the saturated zone under these conditions, the water level indicates the level of the water table; these wells are called *water-table wells*.

Where permeable water-bearing formations (aquifers) are overlain by relatively impermeable formations and the water in the aquifers is confined under hydrostatic pressure, artesian conditions exist. Wells penetrating such aquifers are called *artesian*

* For a water-table map of this area see Illinois State Water Survey Rept. Inv. 17, p. 19.

wells. The water levels in artesian wells stand above the bottom of the confining impermeable bed and may be either above or below the level of the water table at any particular place.

Water-table and artesian systems ideally represent two fundamentally different sets of hydrologic conditions. Commonly, however, the confining layer of the artesian aquifer is only relatively impermeable and thus allows slow transmission of water from the system into adjacent aquifers. This is called a *leaky artesian* condition and it most commonly and nearly always prevails in interbedded unconsolidated deposits with different permeabilities, such as the valley fill and glacial deposits in the East St. Louis area.

Aquifers in the East St. Louis Area

VALLEY FILL

For practical purposes, the only aquifer for large-quantity production in the East St. Louis area is valley-fill material, which includes both alluvium and glacial outwash. Groundwater occurs in the valley fill, with its interbedded layers and lenses of varying permeability, primarily under water-table and leaky artesian conditions. At present, this aquifer furnishes all the groundwater pumped from wells in the valley bottom.

BEDROCK

Bedrock aquifers, although in part capable of producing large quantities of water, are now of negligible importance in the American Bottoms because of the possibility of highly mineralized water at depth, the ready availability of water from shallower valley-fill deposits, and the high cost of deep drilling. In many places on the uplands, however, the bedrock is the only groundwater source available and is tapped for domestic supplies. The shallower bedrock formations in this region are not highly productive, and the deeper ones yield highly mineralized water.

GLACIAL DRIFT

Thin deposits of glacial drift are present on the upland adjacent to the area. This material consists of glacial till overlain locally by 50 feet or more of loess. In some places thin beds of sand and gravel within the till furnish enough water for domestic supplies. These local sand and gravel beds are generally found near the base of the till. They are not persistent and their presence normally cannot be predicted prior to drilling.

GEOLOGIC HISTORY

Paleozoic Era

The present landscape of the East St. Louis area has been produced by processes acting only during relatively recent geologic time. A vast amount of earlier time is represented by the indurated sedimentary rocks that underlie the unconsolidated alluvial fill of the American Bottoms (pl. 4). There is virtually no sedimentary record in this area for the time between the formation of the youngest of these sedimentary rocks (Pennsylvanian) and the advance of Kansan ice during the Pleistocene or glacial epoch. A summary of geologic events is given in table 1.

The bedded rocks of the Paleozoic era beneath the valley fill and in the bluffs of the East St. Louis area rest on the eroded surface of much older (pre-Cambrian) rocks at a depth of over 3800 feet. The Paleozoic seas in which these rocks were deposited as sediments alternately advanced and retreated in the area. The position of the shorelines and the character of the sediments deposited were controlled to some extent by activity in the nearby Ozark area, which was uplifted from time to time, beginning early in the Paleozoic era. The sandy and shaly rocks reflect the washing of sands and muds into the shallow seas, whereas the limestones and dolomites suggest clear seas. No doubt crustal movements were gentle, and neither seas nor highlands were strongly or rapidly modified.

At the close of the Pennsylvanian period the sea withdrew and the area became land. It is likely that the area was never again submerged by the sea, though in other parts of the United States thousands of feet of marine sedimentary rocks were formed dur-

TABLE 1.—SUMMARY OF GEOLOGIC HISTORY

Era	Period	Epoch		Geologic events in East St. Louis area
Cenozoic	Quaternary	Pleistocene	Recent	Shifting of river channel; modification of flood plain; formation of alluvial fans along bluffs.
			Wisconsin	Deposition of valley trains and loess; dissection of valley-train deposits and formation of terraces.
			Sangamon	Weathering and erosion of till and valley-train deposits; reopening of valley.
			Illinoian	Advance of glacier across American Bottoms and onto bluffs at St. Louis; Mississippi River probably maintained course through or under ice.
			Yarmouth	Weathering and erosion of till and valley-train deposits; reopening of drainage through valley.
			Kansan	Advance of glacial ice; deposition of till; possible damming or restriction of Mississippi Valley.
			Aftonian	Weathering, erosion.
			Nebraskan	Advance of glacial ice, which may have reached this area; deposition of valley train.
	Tertiary			Complex series of crustal movements and erosional cycles; establishment of major drainage lines; major cutting of Mississippi bedrock valley.
Mesozoic	Cretaceous			Erosion.
	Jurassic			
	Triassic			
Paleozoic	Permian			Uplift and erosion.
	Pennsylvanian			Periodic submergences by sea with formation of coal swamps during emergent intervals.
	Mississippian			Submergence; formation of shales and thick limestone formations.
	Devonian			Deposition of lime sediments followed by emergence and erosion.
	Silurian			Deposition of limy sediments along outer margin of a great reef belt; later emergence and erosion.
	Ordovician			Continued submergence, with formation of dolomite, shale, and sandstone; intervals of emergence and erosion.
	Cambrian			Prolonged erosion; later submergence and formation of thick beds of sandstone and dolomite.
Pre-Cambrian				Long period of igneous activity, sedimentation, crustal activity, and erosion.

ing the 250 million years after the Pennsylvanian period.

Mesozoic and Tertiary History

The post-Pennsylvanian history of the East St. Louis area is mainly an account of the wearing down of the land by ancient streams during and after periods of crustal uplift. Four cycles of uplift and erosion are recorded in the bedrock surface of western Illinois (Horberg, 1953, p. 39). Each cycle of erosion was initiated by a period of uplift that gave streams more erosive power and caused them to cut into and partially destroy the existing land surface. The oldest erosion surface, because it was involved in all subsequent periods of uplift, has been largely destroyed, but remnants are preserved in the flat upland surfaces of Calhoun County, 25 miles northwest of Alton.

The crustal uplifts produced many drainage shifts. The latest movement probably established the major drainage patterns in essentially their present form, although many segments of river channels were doubtlessly inherited from early courses. Because the Mississippi River between St. Louis and Cape Girardeau cuts across resistant Mississippian rocks, which have been uplifted along the eastern side of the Ozark dome, instead of flowing across the lowland of the softer Pennsylvanian rocks farther east, it is possible that the river was established in its present channel prior to uplift of the dome. Regional structural and geomorphic relationships suggest that the Mississippi Valley is very old. Furthermore, from regional evidence it appears that it may have been cut essentially to its present depth before the advance of Pleistocene glaciers.

Pleistocene Epoch

The advance of continental glaciers into northern United States during the Pleistocene epoch profoundly modified the landscape. Areas actually overridden by the glaciers were blanketed by unsorted rock debris as the ice melted and dropped its load. Beyond the ice front, sediment-laden meltwaters escaped down valleys toward the sea, partially filling them with glacial sand and gravel deposits that became progressively finer downstream. The river flats, kept free of vegetation by frequent glacial flooding, were subject to wind erosion, and great volumes of silt were picked up and transported to the uplands bordering the valleys. The unsorted ice-laid deposits (till), the sorted water-laid material (outwash), and the wind-transported silts (loess) mantle the bedrock in the American Bottoms and adjacent area.

The history of the earlier glacial advances (Nebraskan and Kansan) in the area is obscure, but later glacial events are better documented. The presence of Illinoian till in St. Louis and along the eastern bluffs of the valley indicates that the Illinoian ice, advancing from the northeast, extended across the American Bottoms.

The "clay," "blue clay," and "blue clay and gravel" that are logged in many wells just above bedrock in the Alton–Wood River area may be pebbly glacial till which could be of Illinoian age or older. Because the Illinoian drift is thin, it is unlikely that the valley was completely filled at that time, although drainage was temporarily blocked or restricted so that ponding took place upstream in the Mississippi, Illinois, and Missouri valleys.

The Wisconsin glacial stage in the East St. Louis area was marked by the downstream spread of outwash as valley trains during ice advances in the north and by deposition of loess on the bluffs. Loess is well exposed in the uplands on the eastern side of the valley, particularly in road cuts along Highway 460 between East St. Louis and Belleville where the road first enters the uplands. The loess deposits indicate that the Mississippi valley bottom was covered with extensive valley-train deposits including glacial rock flour from Wisconsin ice sheets. The nearest approach of Wisconsin ice was during the Tazewell substage when the ice advanced into Shelby County, some 75 miles to the northeast.

During one glacial advance, the flood plain at East St. Louis was aggraded to an elevation of about 445 feet. Remnants of this surface are the terraces at Roxana and Wood River and along Cahokia Creek.

Subsequent river downcutting destroyed this surface in all but the northern portion of the American Bottoms. The Recent river scour and reworking have not been complete, however, for the lower section of the valley fill is believed to be partly glacial in origin. Wood fragments found in the lower part of the fill have been dated by the radioactive carbon method as older than 20,000 years, which dates the wood, and presumably the deposits containing the wood, as at least as old as early Wisconsin.

The large boulders commonly encountered at depths of 80 feet or more, which sometimes limit the depth of drill penetration, are probably remnants of Illinoian or older till.

In Recent time the river has scoured and reworked the upper part of the valley fill in migrating across the broad bottomlands. The channel scouring has taken place chiefly during floods when volume and velocity were high. At the same time, spreading floodwaters have deposited silt and clay along the sides of the channel and in backwater areas. In subsiding and low-water river stages, only fine-grained sediments have been transported, and silting has taken place in the channel. The channel migration, cut-and-fill, and flooding have produced complex, heterogeneous deposits which vary in depth (fig. 4). Soundings at Eads Bridge during river flood have indicated river scour as deep as 80 feet (Woodward, 1881, p. 5). This figure is thought to represent the average depth to which the valley fill has been reworked along the Recent meander belt. Below this depth the deposits are glacial outwash material and older alluvium.

The broad alluvial fans found below the bluffs are also of Recent age. They are composed of reworked loess and have been built outward across the valley fill by tributary streams and slope wash.

GEOLOGY AND WATER-BEARING PROPERTIES OF THE BEDROCK

Regional Relations

The river sediments of the American Bottoms are underlain by consolidated sedimentary rocks over 3800 feet thick, as shown by a well completed at the City Sanatorium in St. Louis in 1869 (Broadhead, 1878).

The bedrock formations, dominantly limestone and dolomite with subordinate amounts of sandstone and shale, dip gently northeastward from the Ozark highlands toward the Illinois Basin. In the area of the American Bottoms, minor folds have been superimposed upon the regional structure so that locally the beds may dip in other directions (plate 4). For example, in the southern part of the area a sharp transverse arch produces reversals of the regional dip. The axis of this fold extends from the vicinity of Waterloo in Monroe County in a northwesterly direction through Dupo on the American Bottoms and across the Mississippi at Arsenal Island into St. Louis. The steeply dipping beds on the southern limb of the arch can be seen in the bluffs south of Dupo. The arch is the controlling structure for the accumulation of oil in the Waterloo and Dupo oil fields of Illinois (Bell, 1929). The Florissant dome north of St. Louis is near the trend of the structure.

Mississippian rocks underlie the valley fill in the western part of the American Bottoms, and Pennsylvanian rocks underlie the bottom sediments in the eastern part. The approximate boundary between Pennsylvanian and Mississippian rocks is shown in plate 3. A summary of formations underlying the American Bottoms is given in table 2.

The Mississippi River now follows a channel underlain, beneath the alluvium, by Mississippian limestones. The widening of the Mississippi Valley between Wood River and Dupo is a result of the river's lateral cutting into the easily eroded shales of the Pennsylvanian and Mississippian (Chester) formations upstream from the resistant Mississippian limestones that are at the surface in the Waterloo-Dupo structure.

Lithology of the Bedrock

Most information on the bedrock formations in the American Bottoms has come

Table 2.—Geologic Formations of the East St. Louis Area and Their Groundwater Possibilities

Era	System	Series	Group	Formation	Average thickness	Material	Groundwater possibilities in East St. Louis area
Cenozoic	Quaternary	Pleistocene	Recent alluvium		0–100	Sand, gravel, silt, and clay	Permeable sands and gravels are water-yielding.
			Glacial till, outwash, and loess		10–170	Pebbly clay, sand and gravel, and silt	
	Pennsylvanian				100–400	Shale, sandstone, limestone, and coal	Some of the sandstones and limestones have sufficient permeability to yield water for domestic drilled wells.
	Mississippian	Chester			0–200	Sandstone, shale, and limestone	Some of the sandstones, particularly in lower part of the series (Aux Vases), have moderate permeabilities and are fair-to-good groundwater sources, if close to outcrop area or not too deeply buried.
		Iowa	Meramec	Ste. Genevieve	0–150	Sandy oolitic limestone	Yield water from joints and solution channels. Meramec limestones, particularly St. Louis, are potential water sources north of Alton, in St. Louis, and in sinkhole region south of Stolle.
				St. Louis	200–250	Limestone and dolomite, fine grained	
				Salem	50–100	Dolomite and granular fossiliferous limestone	
			Osage	Warsaw	40–140	Shale and argillaceous limestone	Keokuk-Burlington limestones are less cavernous than St. Louis limestone and therefore not as favorable as a groundwater source except along Dupo arch where limestone is close to surface.
				Keokuk-Burlington	200–270	Cherty crinoidal limestone	
				Fern Glen	45–100	Shaly limestone	
			Kinderhook	Chouteau	10–30	Slightly silty fine-grained limestone	Not water-yielding.
Paleozoic				Hannibal-Grassy Creek	5–50	Dark shale	

Devonian				0–30	Sandy limestone and dolomite	Devonian-Silurian limestone may yield water from joints and solution crevices, but at depth encountered the water is highly mineralized.
Silurian	Niagaran	Bainbridge	Moccasin Springs	20–170	Shaly red limestone	
			St. Clair	30–40	Crystalline pink-speckled limestone	
	Alexandrian		Sexton Creek	20–30	Cherty limestone	
			Edgewood	5–30	Silty dolomite	
Ordovician	Cincinnatian		Maquoketa	140–160	Shale and shaly limestone	Not water-yielding.
	Mohawkian		Kimmswick	75–100	Coarse-grained crinoidal limestone	Kimmswick-Joachim limestone not well jointed or cut by solution channels and not considered a likely groundwater source, even of highly mineralized water.
			Decorah	15–30	Limestone and shale	
			Plattin	100–200	Fine-grained limestone	
	Chazy		Joachim	70–120	Silty dolomite	
			St. Peter	135–155	Clean sandstone, poorly cemented	High permeability, but groundwater highly mineralized.
	Prairie du Chien			850±	Dolomite and sandstone	Most of section is dense dolomite with poor groundwater possibilities. Permeable formations contain highly mineralized water.
Cambrian	St. Croixan			1350±	Dolomite, sandstone, and shale	

Ancient granitic and other crystalline rocks referred to the Proterozoic and Archeozoic eras, called per-Cambrian rocks.

from oil test wells, most of which are drilled to the Kimmswick limestone, the producing formation in the Dupo oil field. Some wells have gone to the St. Peter sandstone; only a few have gone deeper.

A sample-study log of one of the deeper oil tests, drilled 2 miles southeast of Dupo, follows.

Lockwood–Dyroff well 1—NW corner NE¼ sec. 26, T. 1 N., R. 10 W., St. Clair Co. Drilled November 1929. Illinois Geological Survey sample-study set 723, studied by F. E. Tippie. The pre-St. Peter correlations are in part based on a study of this well by John Grohekopf and Earl McCracken, Missouri Geological Survey.

	Depth feet
Pleistocene system	
"Soil"	26
Mississippian system	
Iowa series	
Meramec group	
St. Louis limestone	
"Limestone, white, hard"	45
Limestone, slightly oolitic, finely sandy, white, extra fine	50
"Limestone, white, hard"	90
Limestone, finely sandy, light brown, sublithographic; dolomite, sandy, brown, very fine	95
"Limestone, white, hard"	150
Dolomite, cherty, silty, light gray, very fine	155
"Limestone, white, hard"	165
Limestone, slightly sandy and cherty, buff, very fine	170
Dolomite, partly sandy and argillaceous, light brown, very fine	180
"Limestone, white, brown"	210
Limestone, partly oolitic, slightly cherty, brown, very fine	215
Limestone, dolomitic, brown, extra fine	225
Limestone, dolomitic, cherty, partly oolitic, white to light brown, very fine	235
Limestone, brown, sublithographic	240
Limestone, partly oolitic, dolomitic, white to brown, very fine	265
Dolomite, slightly cherty, brown, very fine	273
Salem limestone	
Limestone, dolomitic, oolitic, slightly cherty, brown, very fine	283
Limestone, brown, lithographic	290
Limestone, dolomitic, oolitic, light brown, very fine	305
Limestone, oolitic, cherty, slightly sandy, light brown, fine	325
Limestone, dolomitic, cherty, brown, very fine	335
Limestone, slightly sandy, mottled gray and brown, medium	345
Limestone, slightly sandy, light brown, medium	385
Limestone, slightly dolomitic, gray, brown, mottled very fine	395
Limestone, slightly dolomitic and sandy, gray, brown, medium, conglomeratic	410
Dolomite, gray, very fine	415
Limestone, dolomitic, brownish, gray, very fine to fine	425
Osage group	
Warsaw formation	
Dolomite, slightly argillaceous, brown, gray, little greenish, very fine	445
Dolomite, very argillaceous, cherty, gray, very fine; shale, dolomitic, gray	460
"Shale, blue, soft"	495
Dolomite and shale as above	500
Keokuk-Burlington limestones	
Dolomite, extra cherty, light gray, very fine, glauconitic	515
Dolomite, argillaceous, extra cherty, gray, very fine, glauconitic	530
Limestone, dolomitic, slightly sandy, white, very fine, partly glauconitic; chert, white, abundant, partly glauconitic	676
Limestone, cherty, white, fine to coarse, crinoidal	681
Fern Glen formation	
Dolomite, very argillaceous, green, grading to shale	690
Limestone, cherty, light brown, reddish, sublithographic	695
Limestone, cherty, white, fine to coarse, crinoidal; shale, calcareous, green	700
Limestone, cherty, white, greenish, very fine to fine, crinoidal; shale, calcareous, green, red at base	715
"Shale, blue, soft"	730
Limestone, argillaceous, slightly cherty, white to red, very fine to coarse, crinoidal; shale, calcareous, red, green	750
"Lime, red, soft"	755
Kinderhook group	
Chouteau limestone	
"Lime, gray, hard"	770
Limestone, white, brownish, sublithographic	785
Hannibal shale	
Shale, dark gray to black, few coarse sand grains at base	798
Silurian system	
Dolomite, silty, slightly cherty, white, little pinkish, very fine	825
Dolomite, cherty, light brown, very fine	830
Limestone, dolomitic, cherty, white, very fine to medium	845
Dolomite, slightly cherty, light brown, very fine	868

	Depth feet
Ordovician system	
Maquoketa formation	
Shale, dolomitic, silty, green, gray, very weak	925
Shale, silty, dolomitic, dark brown	935
Siltstone, calcareous, light brown; dolomite, argillaceous, cherty, silty, gray, very fine	950
Shale, calcareous, brownish gray; and limestone, very argillaceous, brownish gray; little chert	1015
Kimmswick limestone	
Limestone, white to light brown, very fine, little coarse	1020
Limestone, white to light brown, fine to lithographic	1030
Limestone, white, buff, fine to coarse	1065
Limestone, cherty, white to light brown, fine to coarse	1110
Dolomite, brown, very fine	1113
Decorah formation	
Limestone, dolomitic, argillaceous, brown, very fine; little shale, gray	1130
Plattin limestone	
Limestone, slightly cherty, light brown, sublithographic	1140
Dolomite, slightly cherty, light brown, very fine	1165
Dolomite as above; limestone, partly cherty, white, brownish, sublithographic	1240
Limestone, slightly cherty, white to buff, very fine	1260
Limestone, slightly cherty, light brown to white, sublithographic	1285
Limestone, slightly cherty, light brown to white, very fine; little dolomite, dark brown, very fine to base	1325
Joachim dolomite	
Dolomite, light grayish brown, very fine; shale, dolomitic, greenish gray	1335
Dolomite, light gray to light brown, very fine	1385
Dolomite, white, buff, very fine, becoming slightly argillaceous and cherty at base	1410
"Lime, gray, soft"	1425
Shale, green, very weak; dolomite, white, light brown, very fine	1433
Dolomite, argillaceous, brown, gray, greenish, very fine	1440
Dolomite, white to brownish, very fine, finely sandy at base	1473
Shale, dolomitic, finely sandy, gray	1478
Glenwood–St. Peter sandstone	
Sandstone, white to red (iron stain), fine to coarse, incoherent, generally rounded and frosted; little shale, sandy, green at base	1632

	Depth feet
Cotter and Everton formations	
Dolomite, cherty, white, very fine, scattered sand grains, iron stain	1645
Sandstone, white, fine to coarse, iron stain	1650
Dolomite, cherty, white, very fine	1690
Dolomite as above; little sandstone, dolomitic, medium, scattered sand grains	1705
Dolomite as above; little shale, slightly dolomitic, gray	1710
Dolomite, cherty, buff, very fine; sandstone, white, incoherent	1725
Dolomite, cherty, partly sandy, white to buff, very fine	1780
Cotter formation	
Dolomite, partly sandy, white to gray, very fine	1800
Dolomite, cherty, slightly sandy, light brown, very fine	1850
Dolomite, sandy, white to light brown, very fine; chert, banded, oolitic; little sandstone, calcareous, white, fine to coarse	1895
Upper Jefferson City formation	
Dolomite, sandy, white, brownish, very fine; chert, white; sandstone, calcareous, white, fine to coarse; shale, calcareous, gray at base	1985
Lower Jefferson City dolomite	
Dolomite, slightly sandy, very cherty, white, gray, light brown, very fine; chert, bluish, white, translucent	2155
Roubidoux formation	
Dolomite, silty, sandy, gray, buff, very fine; much chert, white, opaque, partly sandy	2240
Sandstone, white, fine to medium, subangular, incoherent; dolomite, as above; little bright green shale at base	2285
Gasconade formation	
Dolomite, white, fine to coarse, scattered sand	2307
Dolomite, very cherty, white, very fine to fine, scattered sand	2450
Dolomite, cherty, partly sandy, white to light gray, very fine to fine	2495
Gunter formation	
Dolomite, very sandy, cherty, white, very fine to fine; sandstone, dolomitic, white, fine	2530
Cambrian system	
Eminence dolomite	
Dolomite, very cherty, white, very fine to fine, scattered sand	2575
Shale, sandy, white, very weak, slightly glauconitic	2580
Dolomite, cherty, partly sandy, white to light brown, very fine to fine	2730

	Depth feet
"Lime, gray, hard"	2740
"Sand, gray"	2764
Potosi dolomite	
Dolomite, cherty, sandy, light brown, very fine to fine with some medium, pyritic	2900
"Sand, white; oil"	2904

The log of the City Sanatorium well in St. Louis (Broadhead, 1878) suggests that the Potosi dolomite, encountered in the lower part of Lockwood-Dyroff well 1, may be underlain by at least 800 feet of Cambrian beds, principally dolomite except for shale beds of the Davis formation and basal Lamotte sandstone.

In the eastern portion of the American Bottoms, wells drilled into bedrock penetrate several hundred feet of shale, sandstone, and thin limestone beds of the Pennsylvanian system and the Chester series (Mississippian) before reaching the massive Mississippian limestones that are near the surface south of Stolle and north of Alton. The sample-study log of a well 1½ miles northeast of Horseshoe Lake illustrates the nature of these upper beds.

Kesl–Kusmanoff well 1—660 feet N line, 330 feet W line, SW¼ SE¼ sec. 12, T. 3 N., R. 9 W., Madison Co. Drilled July 1947. Illinois Geological Survey sample-study set 17178, studied by M. P. Meyer and Heinz Lowenstam. Depths adjusted to electric log and drilling time. Core study from 1215 to 1227 and from 1641 to 1687 feet.

	Thickness feet	Depth feet
Pleistocene and Pennsylvanian systems		
No samples	165	165
Samples not studied	35	200
Shale, gray, carbonaceous, micaceous, weak	30	230
Sandstone, argillaceous, silty, gray, very fine to fine, friable; interbedded shale, sandy, gray, carbonaceous	6	236
Mississippian system		
Chester series		
Paint Creek formation		
Limestone, sandy (very fine), buff, very oolitic, medium to coarse, compact	12	248
Limestone, partly argillaceous, buff, fine to medium, crinoidal	3	251
Shale, calcareous, green, weak	12	263
Limestone, argillaceous at top, brown, medium to coarse, fossiliferous, crinoidal	10	273
Shale, calcareous, mottled red and green, weak	8	281
Yankeetown siltstone		
Siltstone, very cherty, calcareous, white, compact; little sandstone, cherty, calcareous, very fine at top	6	287
Renault formation		
Shale, slightly calcareous, green and gray variegated, weak	3	290
Limestone, sandy (fine), glauconitic, light gray, medium	3	293
Sandstone, calcareous, argillaceous, green, very fine, tight; shale, green, gray to purple, weak	7	300
Siltstone, greenish gray, friable; shale, silty, mottled purple and green at top	10	310
Shale, silty, green, purple, weak; shale, red at base	8	318
Sandstone, silty, light gray to green, very fine to fine, friable	13	331
Shale, red and green variegated, weak	9	340
Shale, silty and sandy, calcareous, green, weak; grading to sandstone, argillaceous, silty, very fine, green	15	355
Shale, as above; pyrite	10	365
Aux Vases sandstone		
Sandstone, slightly calcareous, silty, light gray, very fine, friable	5	370
Sandstone, calcareous, light gray, fine, friable	16	386
Iowa series		
Meramec group		
St. Louis formation		
Limestone, buff, partly sandy, fine to oolitic to lithographic	14	400
Samples not studied	295	695
Limestone, very cherty, buff, fine, oolitic	10	705
Dolomite, very cherty, buff, red speckled, extra fine	50	755
Salem limestone		
Limestone, brown, gray speckled, medium, fossiliferous	5	760
Limestone, dolomitic, grayish brown, fine to medium, fossiliferous, oolitic (*Endothyra*)	15	775

	Thickness feet	Depth feet
Limestone, dolomitic, gray, black specked, extra fine, medium	20	795
Limestone, grayish brown, medium to coarse, fossiliferous	16	811
Osage group		
Warsaw formation		
Dolomite, silty, slightly glauconitic, gray, extra fine; quartz	14	825
Limestone, dolomitic, silty, slightly glauconitic, gray, extra fine; quartz	5	830
Shale, very dolomitic, calcareous, silty, gray, brittle; quartz	27	857
Limestone, argillaceous, silty, gray, fine; quartz	31	888
Keokuk–Burlington limestone		
Limestone, glauconitic, cherty, buff, coarse	14	902
Limestone, very cherty, glauconitic, light buff, medium to coarse	23	925
Samples not studied	125	1050
Limestone, very cherty, white, medium to coarse	15	1065
Fern Glen limestone		
Limestone, dolomitic, silty, cherty, light gray to green, extra fine	30	1095
Limestone, cherty, argillaceous, silty, green, sublithographic	30	1125
Limestone, as above; grading to little shale, calcareous, mottled red and green	27	1152
Kinderhook group		
Chouteau limestone		
Limestone, white to light buff, lithographic	21	1173
Limestone, red, sublithographic	6	1179
Limestone, light green, sublithographic	5	1184
Hannibal–Grassy Creek shale		
Shale, black, weak	26	1210
Shale, silty, gray, weak	5	1215
Shale, brown, tough, sporangites; "Hardin sand" 1 inch at base, argillaceous, coarse, fine, pyritic at base	4	1219
Silurian system		
Niagaran series		
Dolomite, argillaceous, silty, light gray, pyritic	37	1256
Limestone, dolomitic, argillaceous, gray to greenish gray, fine, some red shale partings	22	1278
Shale, calcareous, greenish gray; few limestone streaks, as above	7	1285
Limestone, dolomitic, argillaceous, greenish gray, fine, with pink and red silty shale partings	20	1305
Limestone, silty, argillaceous, red, fine, scattered coarse crinoidal fragments	14	1319
Shale, calcareous, red, brittle	6	1325
Limestone, dolomitic, argillaceous, silty, red, crinoidal	27	1352
Limestone, white to buff, fine to medium, with red crinoidal; streaks siltstone, argillaceous, red	48	1400
Limestone, as above, dolomitic, less crinoidal	6	1406
Alexandrian series		
Kankakee formation		
Dolomite, slightly calcareous, buff, light brown, fine	15	1421
Limestone, slightly glauconitic, white to light gray, medium crystalline, pyritic, very cherty from 1435 to 1452 feet	31	1452
Edgewood dolomite		
Dolomite, calcareous, light brown, fine, sucrose	20	1472
Ordovician system		
Maquoketa shale		
Shale, light greenish gray, weak; streaks siltstone to sandstone, very fine, friable	28	1500
Samples not studied	95	1595
Shale, silty, green, brown speckled, weak	26	1621
Shale, silty, calcareous, green, grayish brown, weak	11	1632
Kimmswick limestone		
Limestone, buff, red speckled, medium	9	1641
Limestone, buff, medium to coarse, fossiliferous, compact, brown and gray shale partings	46	1687

Plate 4 shows representative graphic logs from several deep wells in the American Bottoms.

Groundwater in the Bedrock Formations

No groundwater supplies are being withdrawn from bedrock formations in the American Bottoms, mainly because adequate water supplies of suitable quality are available in the shallower valley-fill material. Groundwater is obtained in St. Louis from wells drilled into upper Mississippian limestones, although the municipal water supply of St. Louis and the major cities of the American Bottoms is obtained from the Mississippi River.

On the eastern upland bordering the valley, water is obtained from sandstones of the Chester series, from sandstones and fractured limestones of the Pennsylvanian system, and from Mississippian limestones. Belleville formerly obtained its water supply from wells drilled 500 to 600 feet deep, into Chester sandstones, but now obtains its supply from East St. Louis.

Beneath the uplands from East Alton to Belleville, Pennsylvanian and Chester sandstones are potential sources of water. Because of their thinness and low permeability, Pennsylvanian sandstones are rarely suitable for other than domestic wells. Mississippian limestones yield groundwater from solution channels and joints. They are potential sources of groundwater mainly between Prairie du Pont Creek and the Mississippi River in the southern part of the area and north and west of Alton in the northern part of the area.

Water obtained from bedrock commonly is too highly mineralized to be acceptable for domestic or industrial use, particularly at depths greater than 370 to 420 feet below ground level on the flood plain and 515 feet below ground level on the uplands (Bowman and Reeds, 1907, p. 56). In general, mineralization increases with formation depth. Analyses of water from bedrock formations in St. Louis County, Mo., show from 4,415 to 11,010.6 ppm total dissolved solids from pre-St. Peter formations and more than 1,000 ppm from the St. Peter at depths below 800 feet (Gleason, 1935). Because the beds dip to the northeast, a given formation generally yields progressively more highly mineralized water in that direction.

The general movement of groundwater is to the northeast, in the general direction of the regional dip of the bedrock formations. Minor structures, as at Dupo, may modify the direction of this movement. The dip of permeable rocks that crop out around the Ozark highlands and the presence of interbedded relatively impermeable shales produce artesian conditions. In the St. Louis–East St. Louis area, the St. Peter sandstone yields water under artesian pressure, although the pressure is insufficient to produce a flowing well. Artesian wells of low yield also have been reported from other formations in the area.

GEOLOGY AND WATER-BEARING PROPERTIES OF THE VALLEY FILL

Bedrock Valley

As shown in the bedrock surface map (fig. 2) and cross sections (fig. 4 and plate 4), the present Mississippi River Valley occupies a deep bedrock valley that has been partially filled by aggrading processes of the river. In much of the area, the bedrock valley floor lies 100 feet or more beneath the bottom of the present valley; in at least one place its depth is over 170 feet (see fig. 3 for thickness of valley fill above the bedrock). Available data indicate that the bedrock valley has steep walls along the present bluff line but that the valley bottom slopes gently toward the middle. In the vicinity of Dupo, the valley narrows as the river crosses resistant Mississippian limestones. Between Dupo and Alton, soft Pennsylvanian sandstone and shale beds form the eastern wall of the bedrock valley. The limestone at Dupo may have resisted downcutting by the river and thus promoted upstream lateral cutting of the Pennsylvanian strata, causing widening of the valley in the middle of the area. Valley widening probably has been aided further by the coincidental location of the weaker beds outside a major bend in the river. The elevation of the bottom of the bedrock valley averages about 310 feet. The bedrock upland bordering the valley on the east ranges in elevation from about 500 feet east of Horseshoe Lake to over 600 feet east of Dupo.

Several types of data suggest that an inner channel, shown within the 280-foot contour lines in figure 2, has been cut at least 20 feet below the average level of the bedrock valley floor.

The log of a test well at Roxana (location A-10) shows 171 feet of valley fill, with bedrock not yet reached. The elevation of bedrock here must be less than 281 feet above sea level. Although there is abundant information from wells in the vicinity of the test hole, reliability of the data concerning depth to bedrock is uncertain. It is likely, however, that bedrock elevation at this location is at least 20 feet below that found in the adjacent area. An oil well between Dupo and East Carondelet penetrated 122 feet of valley fill before reaching bedrock. The bedrock elevation here is 280 feet above sea level, approximately 20 feet lower than in nearby wells. In excavating for the east abutment of Eads Bridge, which connects East St. Louis with St. Louis, bedrock was encountered at 284 feet above sea level. This, too, is approximately 20 feet below the general elevation of the bedrock valley floor.

Another indication of the channel has resulted from seismic work in the area. At several locations in the middle of the valley, bedrock elevations were calculated to be substantially below the elevation of the adjacent bedrock valley floor. Seismic data give elevations for the middle channel that range from 235 feet near the southern border of the area to 260 feet just west of Wood River. It is believed that the indicated 235-foot elevation is too low (possibly by 25 feet) and that the channel floor in this part of the valley is closer to 260 feet above sea level. The basis for this estimate is a Corps of Engineers line of test holes across the Mississippi River four miles to the south, in Monroe County, where the elevation of the channel floor is 256.75 feet. Other seismic stations, apparently over the channel, give elevations of 273, 280, 266, and 263 feet. The linear arrangement of these low elevations and the generally good agreement between seismic results and known elevations tend to confirm the existence of a channel cut below an elevation of 280 feet as far north as Wood River. It is also possible that the channel, at least in the southern part of the area, has an elevation as low as 260 feet. Additional information must be obtained before the exact position and maximum depths of this channel can be determined. On the basis of bedrock elevations given for the Illinois and Upper Mississippi valleys by Horberg (1950), the 280-foot contour line is carried north of Wood River in the bedrock surface map (fig. 2).

Three wells more or less in a line from Monks Mound northeastward also give bedrock elevations somewhat below adjacent areas. These wells record bedrock at an elevation below 290 feet and suggest the presence of a channel—possibly a tributary of the main channel—that swings close to the bluffs north of Caseyville.

In the reach of the Mississippi River known as "Chain-of-Rocks," west of Granite City, the present channel crosses a gently sloping bedrock bench. Along this part of the channel, from approximately a mile north of Merchant's Bridge to a mile north of Chain-of-Rocks Bridge, the river flows partly on bedrock. The shallowness of the water here interferes with river shipping and has led to the construction of Chain-of-Rocks Canal, which serves as a bypass.

Bedrock in the Chain-of-Rocks area is 20 to 80 feet higher than in the remainder of the valley; as a result, the valley fill is thinner by the same amount (fig. 3). As the river is actively eroding the bedrock here, this portion of the bedrock valley is undoubtedly younger than the deeper valley to the east.

The bedrock tributary valleys shown in figure 2 coincide with the present stream valleys. There is, however, a discordance between the bedrock valley and the present Wood River channel between East Alton and Alton where the river enters the American Bottoms. Here the river follows the western side of a mile-wide valley and flows across a spur of Mississippian limestone at an elevation of about 420 feet; half a mile to the east, the bedrock valley is 100 feet deeper and contains about 110 feet of fill.

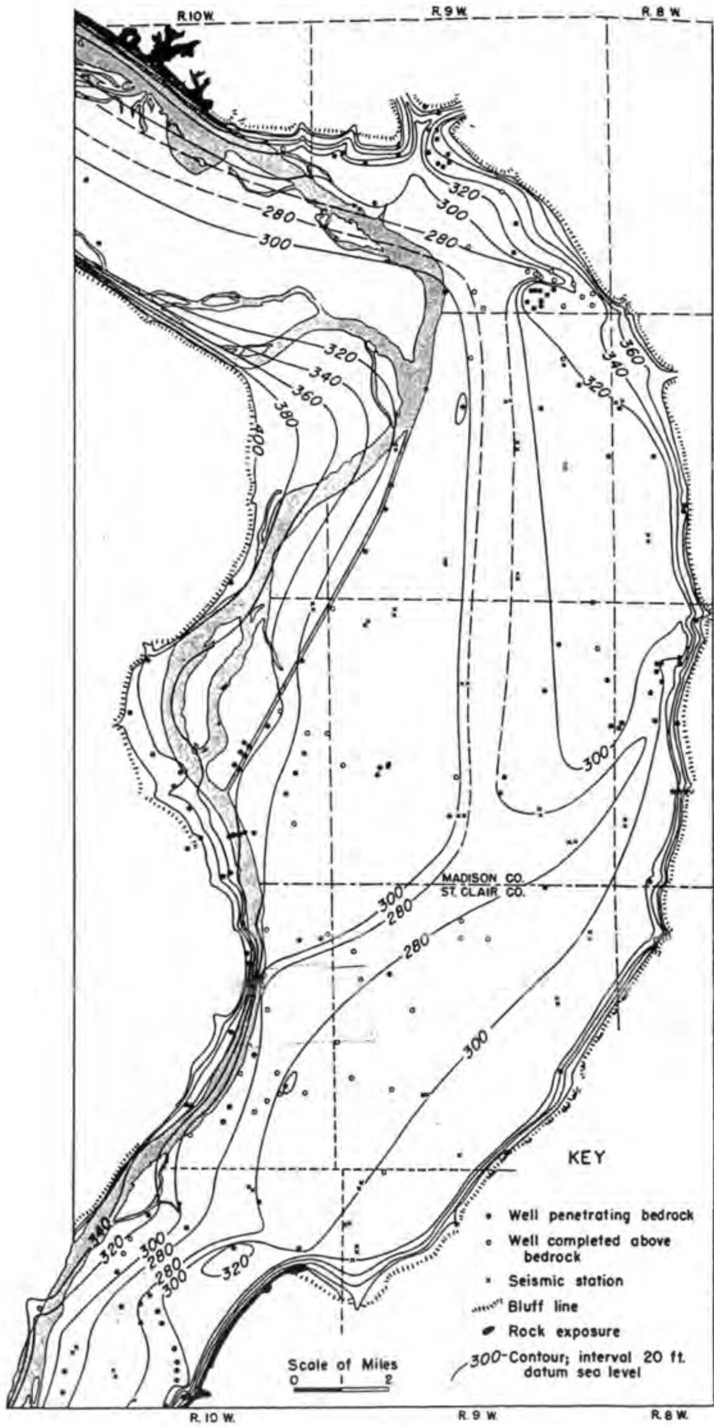

FIG. 2.—Bedrock surface map of the East St. Louis area, Ill.

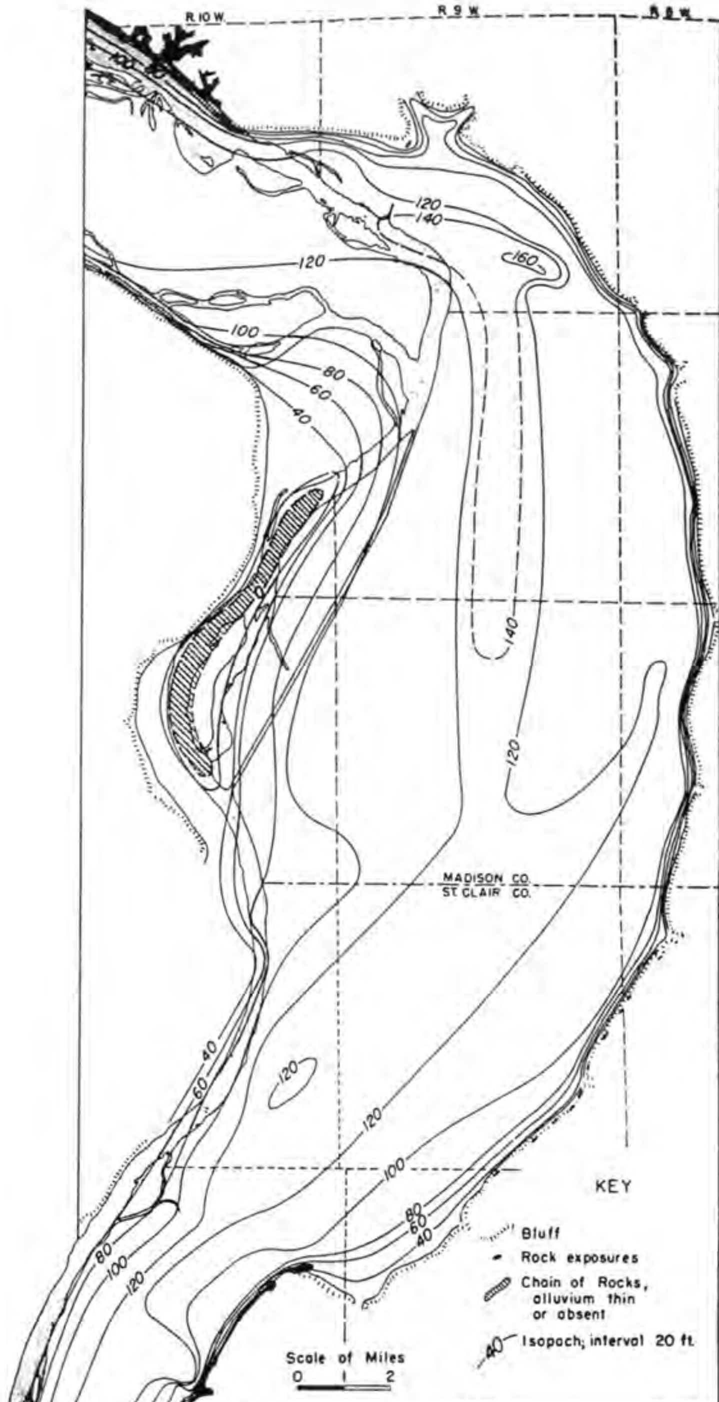

FIG. 3.—Thickness of the valley fill in the East St. Louis area, Ill.

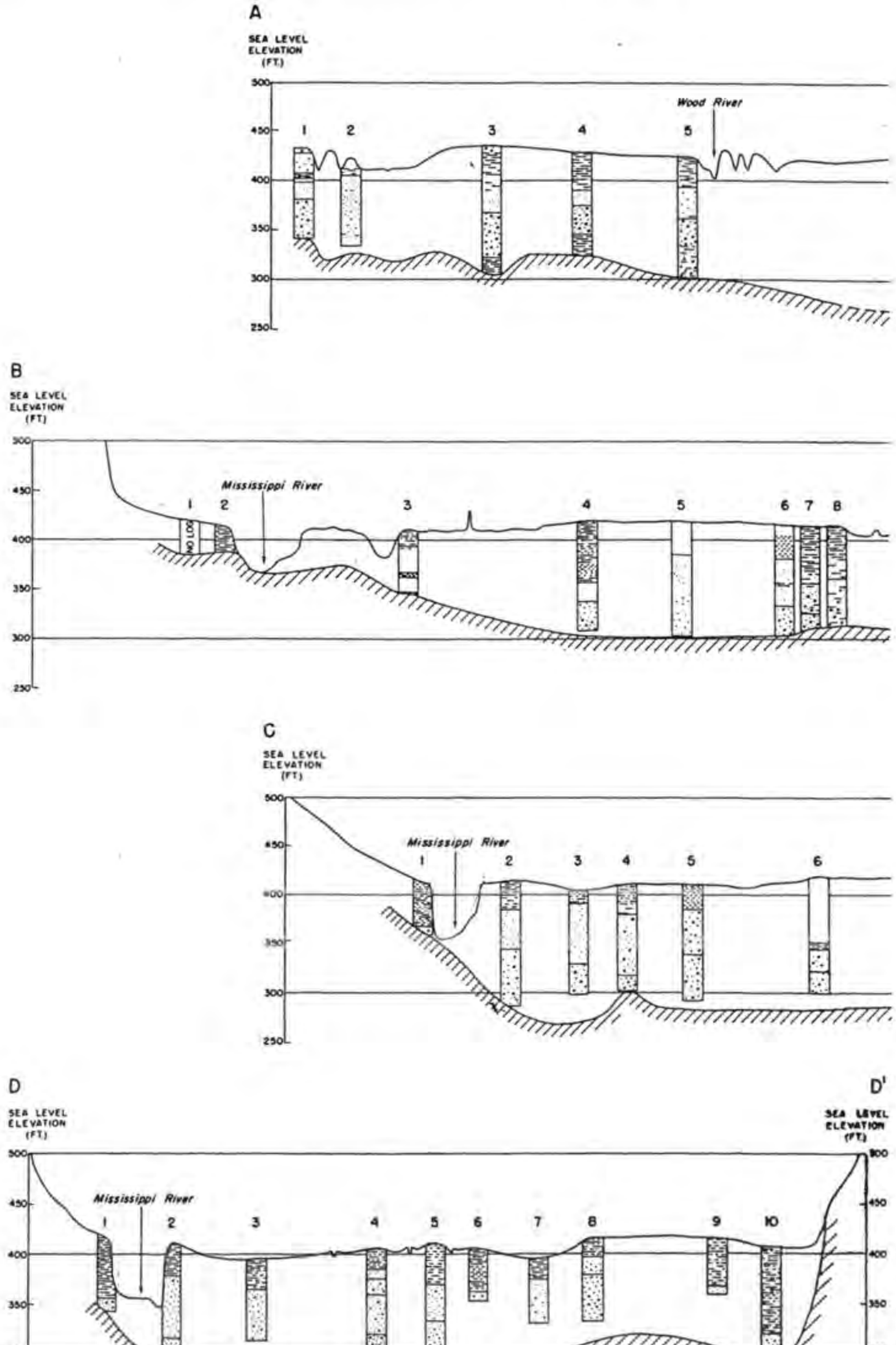

FIG. 4.—Cross sections of the valley fill in the East St. Louis area, Ill.

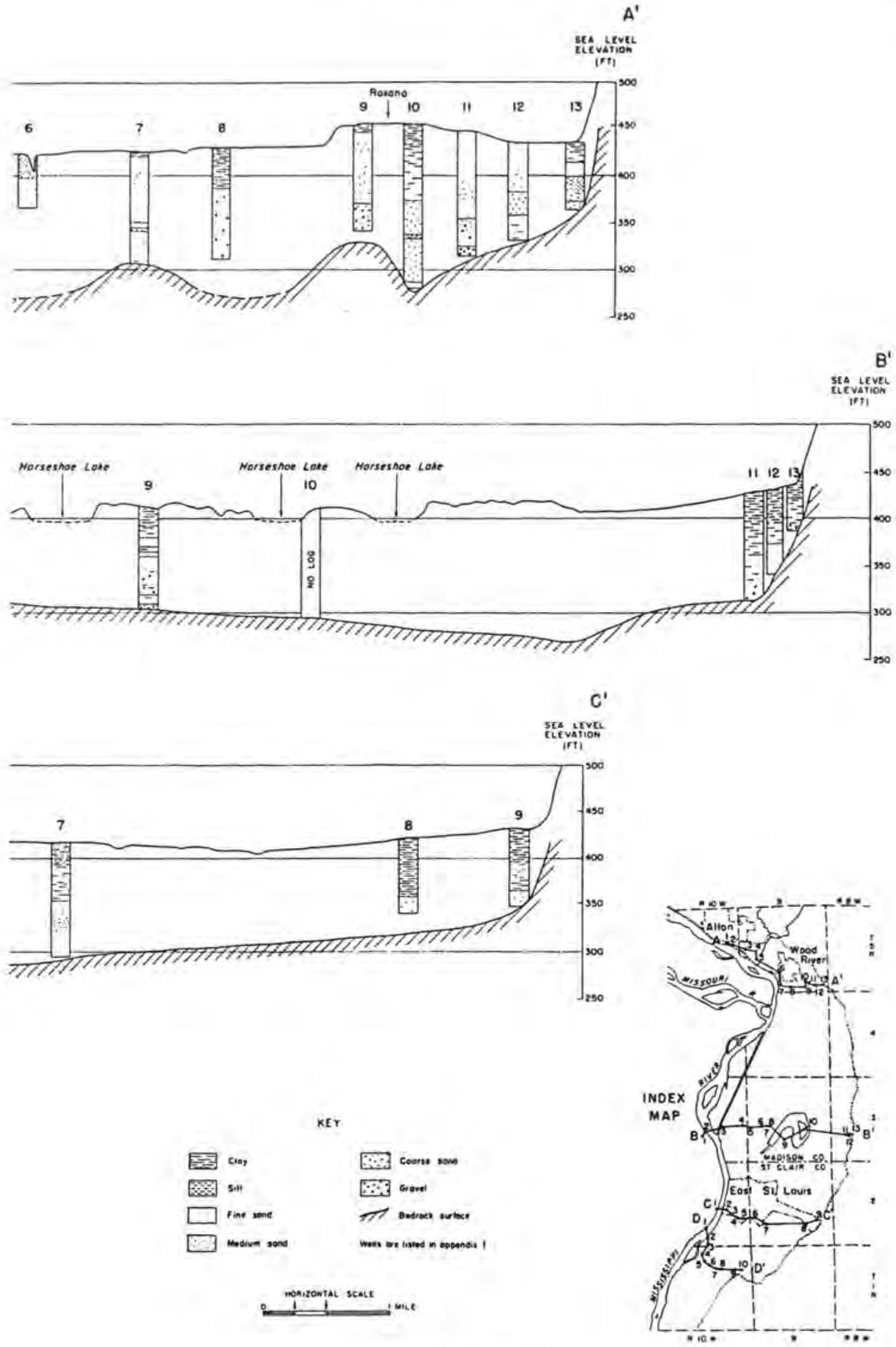
A'
SEA LEVEL ELEVATION (FT)
500
450
400
350
300
250
Roxana
6
7
8
9
10
11
12
13
B'
SEA LEVEL ELEVATION (FT)
Horseshoe Lake
Horseshoe Lake
Horseshoe Lake
9
10
NO LOG
11 12 13
C'
SEA LEVEL ELEVATION (FT)
7
8
9
KEY
Clay
Silt
Fine sand
Medium sand
Coarse sand
Gravel
Bedrock surface
Wells are listed in appendix 1
HORIZONTAL SCALE
0
1 MILE
INDEX MAP
Alton
Wood River
MISSOURI
RIVER
MADISON CO
ST CLAIR CO
East St Louis
MISSISSIPPI

FIG. 4 —(cont.)

Valley Fill

The valley fill of the American Bottoms is composed of Recent alluvium and glacial valley-train material derived from the drainage areas of the upper Mississippi and Missouri rivers. Thickness and cross sections of the valley fill are shown in figures 3 and 4.

Valley-train material is found at the surface in the valley only in terraces in the vicinity of Roxana and Wood River. This material is distinctive in composition and texture (see below). Similar material has been found at depth in a few wells near the terrace, separated from overlying Recent alluvium by a rather marked lithologic break. In most of the area, valley-train material is buried beneath the Recent alluvium.

In most of the American Bottoms, differentiation of valley-train and other alluvial deposits, on the basis of mineralogical and textural characteristics or on lithologic breaks, is not possible. South of the Missouri River mouth, the valley fill contains no apparent discontinuity; valley-train material in this area is apparently mixed with older Missouri River alluvium. These deposits, in addition, have been reworked to varying depths by Recent river scour-and-fill.

GLACIAL VALLEY-TRAIN DEPOSITS

In the Roxana-Hartford area there is a mineralogical difference between the valley-train and Recent alluvial deposits, but south of the Missouri River mouth the valley fill cannot be separated into glacial outwash and alluvial deposits. The sands of the Roxana–Wood River terrace and those in the lower portion of the valley fill at Hartford average 75 to 80 percent quartz, 8 to 15 percent potash feldspar, 5 to 10 percent plagioclase feldspar, and 2 to 6 percent other material. Over 85 percent of the quartz grains are clear and untinted, and the majority are subrounded to rounded. About 10 percent of the quartz grains are pink. Many have flecks of reddish stain in tiny pits on their surfaces. Washing the sand in dilute hydrochloric acid virtually eliminates the pink color of the quartz grains. However, owing to the large proportion of potash feldspar and pink-tinted quartz grains, dry valley-train sands commonly look pink.

The valley-train deposits underlying the terrace at Roxana are texturally quite distinctive. The bulk of the material below shallow depths consists of well-sorted medium-to-coarse sand; median diameters range from .01 inch (.25 mm) to .03 inch (.76 mm). The small amount of gravel present is of granule size (between 4 and 9 mesh).

The sample study of a well at Roxana illustrates the nature of the valley-train material underlying the terrace.

Illinois State Geological Survey test hole 3 (1954) —Roxana Water Works, SE¼ NE¼ SE¼ SE¼ sec. 27, T. 5 N., R. 9 W., Madison Co. Samples studied by R. E. Bergstrom. Est. elev. 445 feet.

	Thickness feet	*Depth feet*
Pleistocene series		
Wisconsin or older Pleistocene		
Clay and silt, yellowish brown, noncalcareous	10	10
Silt and clay, with fine sand, yellowish brown, lumps of pink clay, slightly calcareous	5	15
Sand, fine, dirty, dark reddish brown, calcareous, pink-stained quartz grains	15	30
No samples	5	35
Sand, medium, light reddish brown, calcareous, subrounded grains, rhyolite porphyry, feldspar, graywacke, milky chert	15	50
Sand, medium to coarse, as above	20	70
Sand, fine to very coarse, light brown, dirty, gray silt, coal, mica	20	90
Sand, medium to coarse, light reddish brown, subrounded to subangular grains, abundant feldspar, reddish siltstone and rhyolite porphyry	15	105
Sand, coarse to medium, as above	10	115
Sand, very coarse, as above	5	120
Sand, very coarse, with granule gravel, subangular to angular grains, chert, reddish siltstone, granite, graywacke	5	127
Pennsylvanian system		
Shale, gray and brown	9½	136½

Textural uniformity, which characterizes the deposits of the terrace, does not appear to be a general feature of the valley-train material. Wells near the terrace but on lower levels in the Hartford–Wood

River area pass through deposits that resemble the valley train mineralogically but range from medium sand to pebble gravel. These deposits occur in the lower 20 to 40 feet of the valley fill; in a few wells there is a rather sharp break in composition between them and the overlying alluvium.

The sample study from a well drilled at the Sinclair refinery at Hartford, one mile west of the Wood River terrace, illustrates the nature of the valley-train material beneath Recent alluvium and the lithologic break that separates them.

Sinclair Oil Company well 2 (1952)—150 feet N, 1750 feet E of SW corner sec. 34, T. 5 N., R. 9 W., Madison Co. Samples studied by R. E. Bergstrom. Est. elev. 431 feet.

	Thickness feet	*Depth feet*
Pleistocene series		
Recent alluvium		
No samples	35	35
Sand, very fine, well sorted, olive gray, mollusk shell fragments, abundant mica, coal, wood	35	70
Silt and clay, with fine sand and small gravel, pebbles to ¾ inch, mollusk shell fragments, calcareous	5	75
Wisconsin or older Pleistocene		
Sand, medium to coarse, yellowish brown, dry sample has pinkish cast, grains subrounded to rounded, slightly calcareous	40	115
Sand and pebble gravel, pebbles to 1½ inches in diameter, abundant chert, limestone, graywacke, rhyolite	7½	122½

At the Shell Oil Company loading dock, a mile west of the above location, the lower part of the river fill is also interpreted as glacial valley train. A sample of wood from this material was obtained from a Shell Oil Co. collector well (fig. 5). It is dated as "older than 24,000 years" by the carbon 14 method, which tends to corroborate the valley-train interpretation (Libby, 1954).

South of the Missouri River mouth, valley-train and other alluvial deposits cannot be differentiated. Wells here penetrate, from top to bottom, 10 to 30 feet of surficial silt and clay, silty sand and gravel, and cleaner sand and gravel. At many places coarse bands, generally at depths greater than 75 feet, contain substantial deposits of granule and pebble gravel. Well samples from these zones have numerous pebbles ranging up to 1½ inches in diameter. Some larger pebbles and even large boulders are reported from the lower depths. Median diameters of the water-yielding deposits below the surficial silt and clay range from .008 inch (.22 mm; fine sand) to .08 inch (2.2 mm; granule gravel) in sieved well samples. It is likely that the larger size does not represent the median diameter of the coarsest deposits in the American Bottoms.

Although logs and samples of most wells south of the Missouri River mouth show a general coarsening with depth and give little evidence of a break within the valley fill, it seems reasonable to refer some of the deeper and coarser sand and gravel to glacial origin and the upper material to Recent alluviation. The evidence for this interpretation is: 1) the presence of glacial valley-train material beneath the Wood River terrace and at lower depths at Hartford, as indicated by distinctive composition and carbon 14 dating; 2) studies of present Mississippi River erosion and sedimentation, which show scour up to 80 feet along the present channel but general transportation of mainly fine material; and 3) the presence of extensive deposits containing pebble gravel and boulders, indicative of high velocities and large volumes of water, 100 feet and more beneath the present flood plain.

The coarse deeper deposits are shown by the sample study of a well between Dupo and East Carondelet, in the southern part of the area. In this well the driller reported a thickness of 20 feet of sand, gravel, and boulders below a depth of 75 feet and, below this material, 17½ feet of sand, gravel, and broken rock.

Illinois Geological Survey test hole 2 (1954)—Lutton farm; 4300 feet S of 80° 32′ 30″ N, 5200 feet E of 90° 15′ W, Cahokia Quadrangle, St. Clair Co. Studied by R. E. Bergstrom. Est. elev. 405 feet.

	Thickness feet	*Depth feet*
Pleistocene series		
Recent and older alluvium		
Silt and clay, dark brownish gray	5	5
Silt and clay, with fine sand, dark brownish gray, calcareous, mica	10	15

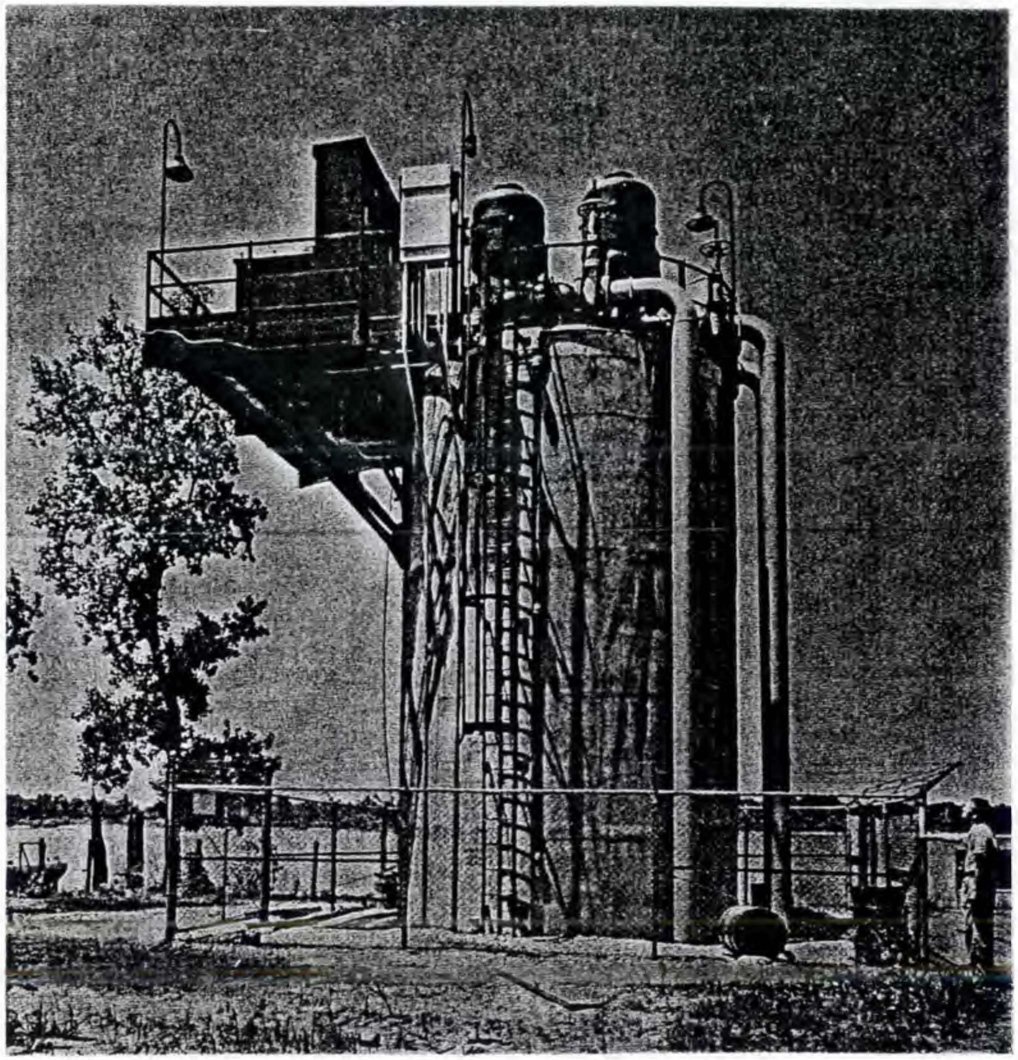

FIG. 5.—Shell Oil Co. high-capacity well at Hartford, Ill. Mississippi River in the background.

	Thickness feet	Depth feet
Sand, fine to medium, dirty, dark olive-gray, mica, wood fragments, coal, tiny calcareous spicules, shell fragments	30	45
Sand, coarse to very coarse, with granule gravel, abundant feldspar, granite, graywacke, chert, and dolomite granules	30	75
Gravel, granule size, with coarse to very coarse sand, quartz, granite, chert, dolomite granules (driller reports boulders)	20	95
Gravel, granule size with broken limestone rock, chert (pebble count of 50 pebbles —15 graywacke and fine-grained basic igneous rock; 12 chert, brown, reddish, and cream-colored; 11 quartz; 3 feldspar; 4 limestone; 4 granite; 1 dolomite); broken rock consists of sharp angular limestone, granite, rhyolite porphyry, and chert	10	105
Broken rock (limestone rubble above solid bedrock?) and granule gravel	7½	112½

The lack of a diagnostic composition in the valley-train material in the southern portion of the American Bottoms may be a result of mixing sediments from the Upper Mississippi Valley with those brought in from the Missouri River drainage basin.

OTHER ALLUVIAL DEPOSITS

Samples of Recent alluvial deposits, obtained from wells at shallow depths close to the present river channel, differ from the valley-train deposits in the Hartford–Wood River area. The sands average 65 to 75 percent quartz, 10 to 13 percent potash feldspar, 12 to 15 percent plagioclase feldspar, and 4 to 7 percent other materials. The quartz grains are dominantly clear, untinted and unstained, and subangular to subrounded. The sand samples commonly look gray, in contrast to the valley-train sands, which look pink.

The grains classified above as "other materials" are chert, limestone, jasper, shale, coal, graywacke, and heavy minerals. The alluvial deposits, like the valley-train deposits, are only slightly calcareous, averaging 3 to 4 percent soluble material by weight.

A further characteristic of the alluvium at Hartford and the upper portion of the valley fill in the area in general is the presence of abundant flakes of mica of the phlogopite and biotite varieties, scattered fragments of pearly mollusk shells, tiny rod-like calcium carbonate spicules, and abundant coal fragments.

The Recent alluvium ranges in texture from clay to granule gravel. The upper 15 to 30 feet is commonly silt and clay with some fine sand. Below this depth the deposits are highly variable, consisting of clean to dirty sand and gravel. These deposits are underlain in most of the area by coarser sands and gravels. Carbon 14 dating of wood obtained from this lower material indicates that in part at least it is older than Recent. Its exact origin is uncertain. It may be older alluvial, valley-train, or reworked valley-train material. The vertical variations in texture contrast with deposits of the Roxana–Wood River terrace.

The sample study from a well at Granite City is typical of many wells on the American Bottoms. It illustrates the occurrence of the upper silt and clay zone, interbedded sand and gravel deposits below the upper fine-grained beds, the coarser material in the lower part, and the lack of a conspicuous break in lithology.

Union Starch and Refining Company (1952)—950 feet S of 38° 42′ 30″ N, 2350 feet E of 90° 10′ W, T. 3 N., R. 10 W., Madison Co. Illinois Geological Survey sample set 23406. Studied by R. E. Bergstrom. Est. elev. 422 feet.

	Thickness feet	*Depth feet*
Pleistocene series		
Recent and older alluvium		
Soil, clay, and silt, dark gray	10	10
Sand, fine to coarse, subangular grains, abundant feldspar, tiny calcareous spicules, coal	30	40
Sand, medium, with granule gravel, as above, mollusk shell fragments	10	50
Sand, fine, with granule gravel, poor sorting, calcareous spicules, abundant dark grains of igneous rocks, ferromagnesium minerals, and coal	10	60
Gravel, granule size, with coarse sand, granules mainly igneous rocks and feldspar	10	70
No samples	10	80
Sand, medium to fine, calcareous spicules, subangular grains, coal	10	90
No samples	5	95
Sand, very coarse to coarse, with granule gravel, pinkish cast, abundant pink-stained quartz grains, subangular to subrounded grains	15	110
Sand, medium, well sorted, pink, subrounded to subangular grains, abundant pink feldspar	5	115

In figure 6, four mechanical analyses plotted as cumulative frequency curves illustrate the consistency of the valley-train deposits of the Roxana–Wood River terrace compared with deposits of other parts of the American Bottoms. The good sorting of the terrace deposits is indicated in the upper two curves by their steepness. The consistency of the textures with depth is shown by the close spacing of the curves representing different depths. The lower curves, of sam-

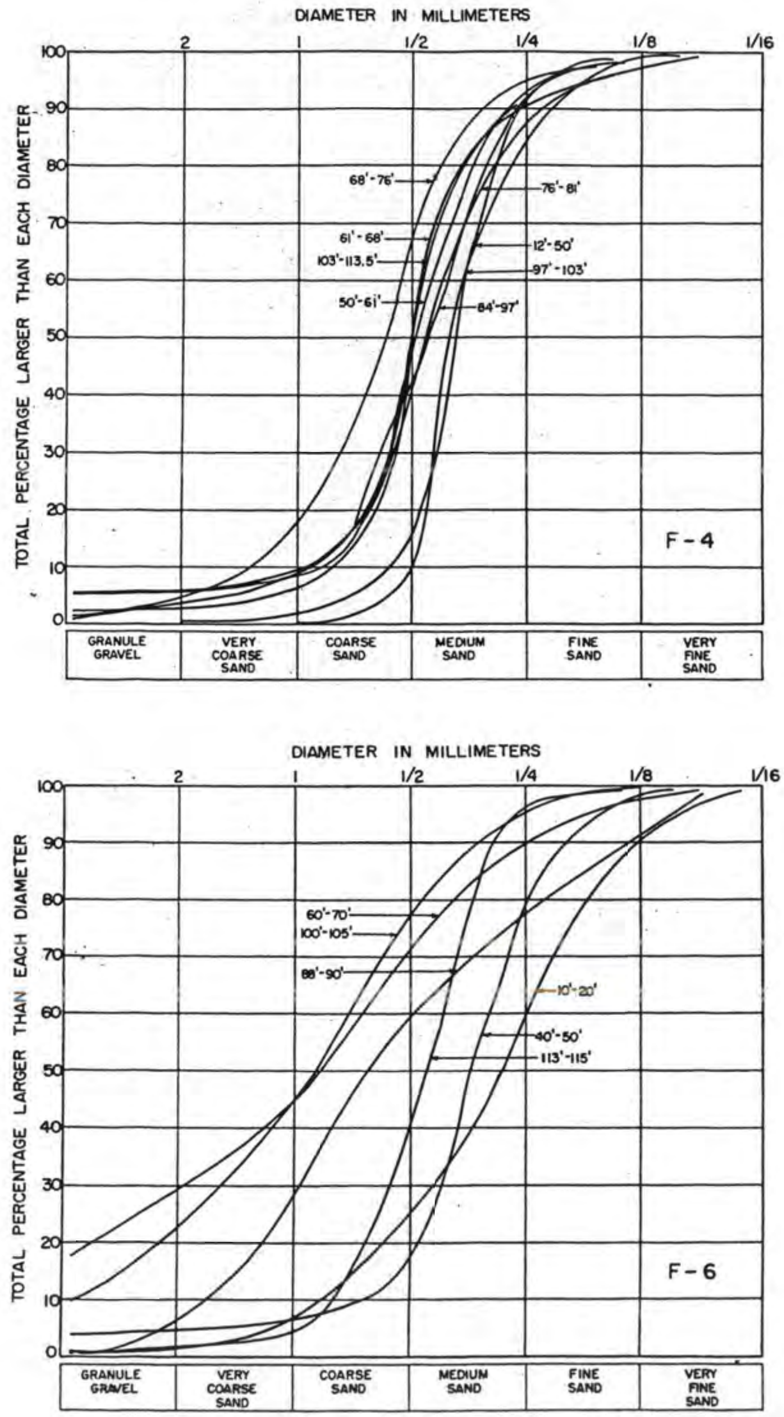

FIG. 6.—Cumulative frequency curves showing mechanical composition of well samples. Wells F-4 and F-2 (top, above and right) are located on terrace at Roxana and Wood River. Wells F-6 and F-9 are on flood plain at Granite City and Monsanto, respectively. Figures beside curves are depths of sample in well. Note good sorting (shown by steepness of curve) and similarity of textures at different depths in well (shown by close grouping of curves) of wells F-4 and F-2 on terrace.

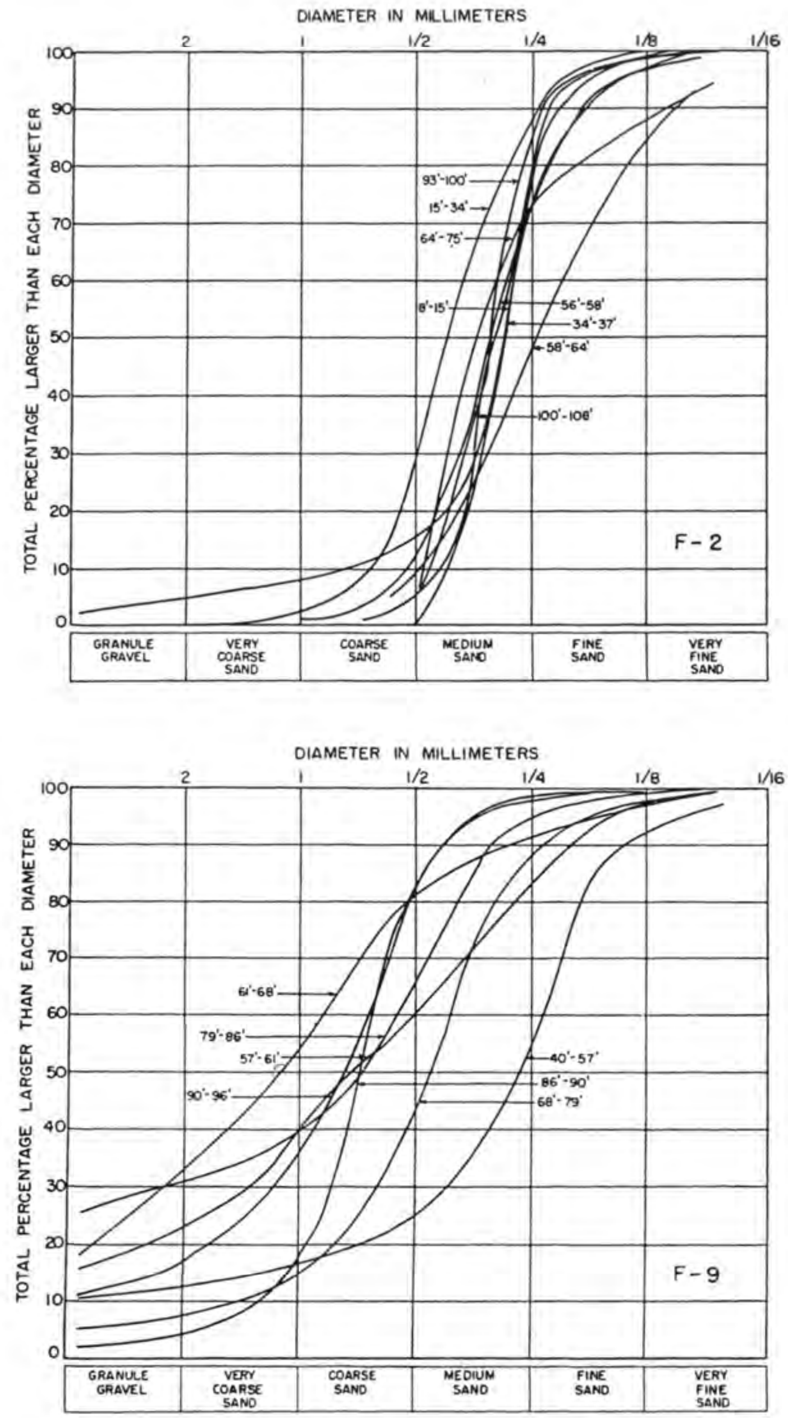
DIAMETER IN MILLIMETERS
2
1
1/2
1/4
1/8
1/16
TOTAL PERCENTAGE LARGER THAN EACH DIAMETER
100
90
80
70
60
50
40
30
20
10
0
93'-100'
15'-34'
64'-75'
8'-15'
56'-58'
34'-37'
58'-64'
100'-108'
F-2
GRANULE GRAVEL
VERY COARSE SAND
COARSE SAND
MEDIUM SAND
FINE SAND
VERY FINE SAND
DIAMETER IN MILLIMETERS
61'-68'
79'-86'
57'-61'
90'-96'
40'-57'
86'-90'
68'-79'
F-9

Fig. 6.—(cont.)

ples from wells at Monsanto and Granite City, indicate poorer sorting, greater variation in texture with depth, and occurrence of fairly coarse deposits in the lower part of the valley fill.

The results of mechanical analyses of well samples (appendix 2) must be accepted with caution. The valley-fill material is highly variable throughout, so a small sample is at best characteristic only of the sediment in its immediate vicinity. In addition, these are not undisturbed samples. Some have been collected from wells drilled with cable tool rigs, some from wells drilled with rigs of the reverse rotary type, and others from wells dug with a clam-shell type digger. Most of the samples were collected by the driller or an assistant, so the conditions of collecting are not known. The evidence that these analyses present, therefore, is only suggestive.

DISTRIBUTION OF VALLEY-TRAIN AND OTHER ALLUVIAL DEPOSITS

Alluvium of Recent age probably comprises the major portion of the valley fill, although its thickness varies considerably. Beneath the terrace it is absent and valley-train material is at the surface, whereas in some areas of shallower bedrock, as in the vicinity of Chain-of-Rocks, Recent alluvium extends to bedrock.

In general, the thickness of Recent alluvium is a measure of the scouring effect of the river since the latest Pleistocene glaciation. Deep scouring occurs in the spring when there are floods and in the winter when thick ice jams cause the river to deepen its channel in order to pass beneath the ice. Soundings taken through the river ice prior to the construction of Eads Bridge indicate that at least 80 feet of channel deepening (scour) takes place (Woodward, 1881, p. 5). The effect of this scour (in combination with channel migration) has been to produce an upper blanket of Recent alluvium resting on older deposits, some of them glacial valley-train. The Recent alluvium coarsens with depth as a result of successive periods of scour and deposition. the largest particles settling out first. Coarsening is also general in the older material, below the Recent alluvium. The uppermost portion of the alluvium contains only fine-grained material; its thickness is further increased at the surface by deposition of silt and clay from floodwaters that cover the area after the channel has migrated to a new position. The cross sections (fig. 4) and cumulative frequency curves (fig. 6) illustrate the increase in grain size from the surface down.

The deposits of the Roxana–Wood River terrace and those in the area just south of Alton are exceptions to the general textural pattern of the fill. Several wells just south of Alton (wells A-3 and A-4, fig. 4) penetrate sections of "clay," "clay and silt," and "clay and gravel" at the bottom of the valley fill. The maximum thickness of the material is 25 feet. These deposits may be Illinoian or older. No samples of the lower material could be obtained for study, so the origin of the material is uncertain.

WATER-YIELDING CHARACTERISTICS

The valley-train material underlying the terrace at Roxana and Wood River is well-sorted medium-to-coarse sand throughout most of its thickness, whereas the complex alluvial deposits in other parts of the American Bottoms generally show poor sorting in the upper part and an increase in coarseness with depth. Permeabilities in these deposits are therefore greatest in the deeper parts, especially where clean coarse sand and gravel occur. The sand and gravel, 20 to 50 feet thick at many places, appear to be the most permeable of any deposits in the area, surpassing the finer material of the terrace. From the standpoint of actual well yield, however, the terrace deposits may be as favorable an aquifer as the coarser sand and gravel—despite lesser permeability—because they are considerably thicker, averaging more than 80 feet.

Evaluation of pumping tests in progress in the American Bottoms by the State Water Survey will yield quantitative data on permeabilities and transmissibilities of the deep coarse sand and gravel and the Roxana–Wood River terrace deposits.

The valley fill in some areas, however, such as north of Horseshoe Lake, is composed of fine-to-medium sand and silt throughout most of its thickness and has poor groundwater possibilities. Thus the valley-fill deposits, except for those on the terrace, are characterized not only by excellent groundwater supply potentialities but by inconsistency. The terrace material, on the other hand, probably is somewhat less permeable but is a thicker and more consistent aquifer, although somewhat restricted in lateral extent.

Some drillers in the area drill to bit refusal and then set screen in the lower 10 to 40 feet of the section. However, good water-yielding beds, in Recent alluvium as well as in glacial outwash, are not everywhere restricted to the lower part of the section. In many instances shallower deposits, which might increase the yield of the completed well, are cased off. In the drilling of new wells it is recommended that, where maximum yield and specific capacity are desired, setting screen opposite the shallow permeable deposits as well as opposite the deep permeable deposits be considered.

Groundwater Recharge

The principal means of recharge of groundwater in the valley fill are seepage from rainfall and floods, and percolation from the Mississippi River and its tributaries. Rainfall is probably the most important source for the area as a whole, although where heavy pumpage is concentrated near the river the recharge from the river itself is undoubtedly great. The effectiveness of recharge from both rainfall and floodwaters is significantly influenced by the nature of the material in the upper portion of the valley fill, which throughout most of the area is 10 to 30 feet of silt and clay. This fine-grained material is usually not so impermeable as to prevent appreciable recharge. There is very little runoff because of the low relief; hence most of the rainfall either evaporates or seeps into the soil. Recharge from floodwaters is undoubtedly much less at present than it has been in the past because of the extensive flood-control program, which is continually being expanded. Where floods do occur they probably result in appreciable recharge.

The recharge from tributary streams that cross the valley flat is probably seasonal for the most part. As the gradient of the streams is very low, the normally slow-moving water can carry only the finest material. The bottoms of the channels probably are covered with a relatively thick deposit of mud, which permits only very slow movement of water into the material below. After periods of prolonged rains in the upland watershed areas, the streams rise, their velocities are greatly increased, and they probably scour their channels sufficiently to remove the impermeable mud, which temporarily permits more rapid recharge. Under natural conditions the streams would be subject to considerable periodic flooding, but man-made changes have prevented most of the floods. Courses have been straightened, channels deepened and widened, and levees constructed. As a result, the tributary streams are not now as large a source of recharge as they once were.

The Mississippi River is an important source of recharge where heavy pumpage has lowered the water table below the level of the river (Bruin and Smith, 1953). Lowering the water table causes the development of hydraulic gradient away from the river and toward the area of pumpage. During high-water stages the hydraulic gradient is increased, which in turn increases the effectiveness of recharge.

Although many areas of the river channel are normally floored with silt, which limits water infiltration, permeable sandy areas are probably present in the channel. Observations on the Mississippi indicate that even in comparatively straight reaches, the thread of the stream moves from one side of the channel to the other, producing shoals and deeps and accompanying differences in bottom deposits. Therefore even under ordinary conditions some groundwater recharge from the river is likely. During high-water stages, when the river scours its channel, recharge conditions are improved.

The only area of notably unfavorable conditions for recharge is west of Granite City where the bedrock lies at a shallow depth and the coarse deposits generally found in the lower part of the fill are either very thin or missing (fig. 4, B-B′).

Local Groundwater Conditions in the American Bottoms

The occurrence of thick clean deposits of deep sand and gravel over wide areas in the American Bottoms has been partly responsible for the heavy industrial development of the area. Over 100 million gallons of groundwater a day is consumed by industries. Monsanto, Granite City, and Wood River–Roxana–Hartford are the major pumpage centers (Bruin and Smith, 1953). Major cones of depression have been produced by heavy pumpage in these areas.

Despite the present heavy industrial groundwater consumption, it is likely that much more groundwater could be available if industrial expansion takes place in favorable but unexploited areas, particularly near the river where recharge might be induced.

Although the variability of the valley fill and deficiency of well data in many parts of the American Bottoms make it impractical to show groundwater supply potentialities on a map, a summary of groundwater conditions in the various parts of the American Bottoms follows.

Alton–Wood River–Hartford–Roxana area.—Graphic sections showing the lithology of valley-fill material in the area are given in figure 4. They show that the bedrock surface is quite irregular. The eastern part of the section, beginning with well A-9, shows the nature of the terrace material. It is dominantly medium-to-coarse sand, with little gravel, and fairly uniform from top to bottom. Eastward the terrace surface becomes lower and the deposits are finer and contain more silt.

Clean deposits of sand and gravel are found at depths below 50 feet from Alton southeast to Hartford. Many wells in this belt have encountered clay as much as 25 feet thick overlying the bedrock, but above this material coarse sand and gravel are found. The river-front area from Alton to Hartford is geologically favorable for further groundwater development.

Area along Cahokia diversion channel and Chain-of-Rocks Canal.—The valley-fill material in this area has been investigated in connection with U. S. Army Corps of Engineers channel and levee projects (unpublished data, U. S. Army Corps of Engineers, St. Louis district). Borings penetrated thick deposits of clean sand and gravel, except near the southern end of Chain-of-Rocks Canal, west of Granite City, where the bedrock is shallow and coarse deposits are thin (fig. 3).

Area north and east of Horseshoe Lake along bluffs.—The area just west of the bluffs from the vicinity of Poag south to the Madison County line is the site of the Edwardsville, Troy, and Collinsville wells. The bedrock rises sharply at the eastern margin of the flood plain, but from one-half to three-fourths of a mile west of Highway 157, which follows the base of the bluffs, the bedrock floor is reached at a depth of 100 feet or more. Deposits of clean sand and gravel 20 to 40 feet thick have been penetrated. The coarseness of these deposits decreases toward Horseshoe Lake. Some of the coarsest sand and gravel studied came from the valley fill near the bluffs.

Because of the thick, deep sections of clean sand and gravel, this area is considered geologically favorable for greater groundwater development.

Granite City–Madison area.—The lithology of the valley fill in the Granite City area is shown in figure 4, B-B′. The bedrock surface slopes eastward. Bedrock is exposed in the river channel west of Cabaret Island during low-water stages, but between Granite City and Horseshoe Lake it is about 115 feet below the surface of the flood plain. Deposits of clean sand and gravel 20 to 35 feet thick are encountered at the base of the fill at Granite City and Madison. These deposits become finer toward the east, and within half a mile of Horseshoe Lake they pass into dominantly sand and silt deposits unfavorable for industrial groundwater supplies.

Central belt.—A north-south belt 3 to 4 miles wide, extending from a point opposite the mouth of the Missouri River south to the Madison County line, does not appear to be favorable for the development of large supplies of groundwater. The valley fill in this belt is fine-grained material, apparently of low permeability. The nature of this material is illustrated by well B-9 in figure 4.

East St. Louis.—The deepest part of the bedrock channel appears to pass under East St. Louis, not far east of the eastern pier of Eads Bridge, where the bedrock surface is 284 feet above sea level. Wells in East St. Louis and east of the city were completed in clean sand and gravel of high permeability 20 feet or more thick. To the north, well logs at the National City stock yards record mainly medium-to-coarse sand, with little gravel.

Monsanto–Cahokia–Prairie du Pont–Dupo area.—The southern part of the area, south of East St. Louis, is highly favorable for industrial supplies of groundwater. Monsanto and Cahokia are already heavily developed, but the area to the south, with the same possibilities, has not been exploited. Coarse, permeable sand and gravel deposits are present throughout the area, as indicated by industrial wells and Corps of Engineers levee borings. C-C′ and D-D′ of figure 4 illustrate the lithology of the valley-fill materials and the nature of the bedrock surface. The presence of coarse deposits close to the river in this area favors recharge from the river, if water levels on the flood plain are sufficiently lowered by pumpage.

CONCLUSIONS

Certain generalizations on present and future development of groundwater supplies in the American Bottoms can be made from the preceding discussion.

1. Coarse alluvial and valley-train sands and gravels, generally concentrated near the base of the valley fill, have high permeabilities and are the most favorable deposits for yielding industrial supplies of groundwater.

2. The medium-to-coarse sands that underlie the terrace at Wood River and Roxana are excellent deposits for yielding industrial supplies of groundwater, although they are somewhat restricted in lateral extent and may have slightly lower permeabilities than the coarser deposits in other parts of the American Bottoms.

3. Because the terrace deposits are consistently finer in texture than are the deeper sand and gravel deposits elsewhere in the area, wells situated on the terrace in the Roxana–Wood River area would require finer gravel packs and screens for maximum efficiency than wells constructed in the lower coarse sand and gravel at East St. Louis, Granite City, Monsanto, and Cahokia. Median diameters of the terrace material range from .01 to .03 inches; median diameters of the coarse sand and gravel, .02 to .08 inches.

4. Because of the variable nature of the alluvium over much of the American Bottoms, highly permeable zones are present in some places at depths as shallow as 60 to 70 feet. The practice of setting screens only in the lower portion of wells may result in failure to take full advantage of the water-yielding capabilities of these shallower permeable zones. Therefore, where maximum yield and highest specific capacities are desired, consideration should be given to setting screens through all zones of high permeability that are of sufficient depth that the screens will not be exposed to air as a result of drawdown from heavy pumpage.

5. Greater appreciation of the variability of the valley fill during design and construction would lengthen the life and improve the efficiency of wells. Wells in the American Bottoms have been found to have a much shorter life expectancy than those in the State as a whole. The principal causes of well failures in the area are screen-clogging and the filling of wells with sand. Screen-clogging is partly chemical and partly mechanical (Bruin and Smith, 1953). Sand-clogging will be reduced if careful consideration is given to the texture ranges throughout the screened intervals. The texture of the alluvium may vary greatly within a few feet vertically, making it impossible to select a screen with one slot size optimum for the entire screened interval. Therefore, consideration should be given to the use of

composite screens made up of sections of different slot sizes. The life of many wells also would be increased by the use of carefully constructed gravel-packed wells. Clogging of screens and filling of wells with sand will be at a minimum if the gravel pack surrounding the screen has the proper textural relationship to the material in the adjacent alluvium. A uniform-grain-size gravel with a median grain size between 5 and 10 times the median grain size of the water-yielding formation has been found to give excellent results (Smith, 1954, p. 15).

6. The valley fill appears to be unfavorable for yielding industrial supplies of groundwater in portions of the American Bottoms where glacial alluviation and Recent river cut-and-fill have produced silt and fine sand extending almost to bedrock. Such conditions are believed to be present in a wide belt extending from opposite the mouth of the Missouri River to the area south of Horseshoe Lake.

7. Owing to the shallow permeable deposits along the present Mississippi channel, conditions are probably favorable for recharge from the river in most areas along the river where the water table is sufficiently lowered by pumpage. Induced recharge from the river becomes especially important in the face of increased demand for groundwater because flood-control measures have restricted the normal spreading of floodwater over the American Bottoms, formerly an important factor in recharge.

8. Increased groundwater development appears possible in three areas where present withdrawals are small compared to the potentialities believed to exist. These areas are: 1) between the eastern bluffs and Horseshoe Lake from the Madison–St. Clair county line north to Roxana; 2) along the Mississippi River near the mouth of Wood River; and 3) the East Carondelet–Dupo area in the south, extending to an area east of Cahokia.

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APPENDIX 1

PARTIAL LIST OF WELLS IN THE EAST ST. LOUIS AREA

A-1 Corps of Engineers, boring, 1948. 400 feet E of center W line, sec. 13, T. 5 N., R. 10 W., Madison Co. Elev. 434 feet. Total depth 92 feet, bit refusal. Engineer's field log.

A-2 Owens Illinois Glass Co. well 9. Thorpe Concrete Well Co., 1950. Center of NE ¼ SW¼ sec. 13, T. 5 N., R. 10 W., Madison Co. Elev. 422 feet. Total depth 88 feet. Finished in sand. Driller's log.

A-3 Alton Boxboard Co. test hole H. Layne-Western Co., 1944. 2400 feet E, 1300 feet N, SW corner sec. 18, T. 5 N., R. 9 W., Madison Co. Elev. 436 feet. Total depth 131 feet, on rock. Driller's log.

A-4 Alton Boxboard Co. test hole J. Layne-Western Co., 1944. 200 feet N, 200 feet W, SE corner sec. 18, T. 5 N., R. 9 W., Madison Co. Elev. 428 feet. Total depth 104 feet, on rock. Driller's log.

A-5 Illinois Power Co., Wood River Power Station, test boring 4, 1947. 1500 feet N, 1900 feet E, SW corner sec. 20, T. 5 N., R. 9 W., Madison Co. Elev. 425 feet. Total depth 123 feet, bit refusal. Driller's log.

A-6 Shell Oil Co., loading dock, well W-1, Ranney Well Co., 1952. 2600 feet N, 2700 feet W, SE corner sec. 33, T. 5 N., R. 9 W., Madison Co. Elev. 425 feet. Total depth 118 feet, on bedrock. Driller's log.

A-7 International Shoe Co., Layne-Western Co., 1951. 2200 feet N, 800 feet E, SW corner sec. 34, T. 5 N., R. 9 W., Madison Co. Elev. 429 feet. Total depth 117 feet, finished in clay. Driller's log.

A-8 Shell Oil Co. well 15, Thorpe Concrete Well Co., 1927. 2110 feet from W line, 278 feet from N line SW ¼ sec. 35, T. 5 N., R. 9 W., Madison Co. Elev. 454 feet. Total depth 112 feet 11 inches, finished in coarse sand and gravel. Driller's log.

A-9 Shell Oil Co. well 54, Thorpe Concrete Well Co., 1949. 1900 feet S, 1000 feet W, NE corner sec. 35, T. 5 N., R. 9 W., Madison Co. Elev. 446 feet. Total depth 131 feet, finished in gravel. Driller's log.

A-10 Shell Oil Co., Wood River, test hole 6, Layne-Western Co., 1942. 1100 feet S, 2300 feet E, NW corner sec. 35, T. 5 N., R. 9 W., Madison Co. Elev. 452 feet. Total depth 171 feet, finished in sand. Driller's log.

A-11 Shell Oil Co. test hole 10, Thorpe Concrete Well Co., 1946. 2200 feet S, 1250 feet E, NW corner sec. 36, T. 5 N., R. 9 W., Madison Co. Elev. 435 feet. Total depth 102 feet, finished in sand. Driller's log.

A-12 Shell Oil Co., Recreation Center test well, Roxana, Ill., Harold L. Watson Drilling Co., 1950. 2900 feet N, 1750 feet W, SE corner sec. 36, T. 5 N., R. 9 W., Madison Co. Elev. 270 feet. Total depth 71 feet. Driller's log.

B-2 City of St. Louis River Front Project D. H. 102, 1951. 5350 feet S of 80° 42′ 30″ N, 100 feet E of 90° 12′ 30″ W., St. Louis Co. Elev. 414 feet. Total depth 22.7 feet, bit refusal. Engineer's field log.

B-3 Corps of Engineers, Chain-of-Rocks lock site, boring H-1, 1941. 2600 feet from N line, 240 feet from W line, sec. 23, T. 3 N., R. 10 W., Madison Co. Elev. 412.4. Total depth 73.7 feet, finished in gray limestone. Engineer's field log.

B-4 Hoyt Metal Co., Granite City, Thorpe Concrete Well Co., 1936. 4200 feet S of 38° 42′ 30″ N, 2600 feet E of 90° 10′ W, T. 3 N., R. 10 W., Madison Co. Elev. 421 feet. Total depth 111 feet 6 inches, finished in boulders and sand. Driller's log.

B-5 Granite City Steel Co. well 21, Harold L. Watson Drilling Co., 1946. 4700 feet S of 38° 42′ 30″ N, 5400 feet W of 90° 07′ 30″ W, T. 3 N., R. 9 W., Madison Co. Elev. 421 feet. Total depth 116 feet, finished in sand. Driller's log.

B-6 St. Louis Gas and Coke Co. well. SW ¼ NW ¼ sec. 20, T. 3 N., R. 9 W., Madison Co. Elev. 417 feet. Total depth 114 feet, finished in sand and gravel. Driller's log.

B-7 Koppers Co. test hole 3, Layne-Western Co., 1948. 1900 feet S, 1400 feet E of NW corner sec. 20, T. 3 N., R. 9 W., Madison Co. Elev. 416 feet. Total depth 104 feet, on rock. Driller's log.

B-8 Koppers Co. test hole 4, Layne-Western Co., 1948. 1800 feet S, 2900 feet W, NE corner sec. 20, T. 3 N., R. 9 W., Madison Co. Elev. 417 feet. Total depth 103 feet, finished in sand and boulders. Driller's log.

B-9 Illinois Geol. Survey test hole 1, Charles M. Hayes, 1954. 125 feet E, 250 feet N, SW corner NW ¼ sec. 28, T. 3 N., R. 9 W., Madison Co. Elev. 413. Total depth 111 feet, finished in bedrock. Samples studied by R. E. Bergstrom. Sieve analysis.

B-10 Neidringhous-Sullivan well 2, 1932. 1600 feet from S line, 1825 feet from E line, sec. 22, T. 3 N., R. 9 W., Madison Co. Elev. 411. Total depth 1105, finished in Hannibal shale. Driller's log.

B-11 Village of Troy test hold 3, Layne-Western Co., 1953. Approx. 100 feet N, 3310 feet W of SE corner sec. 20, T. 3 N., R. 8 W., Madison Co. Elev. 430 feet. Total depth 115 feet, finished in shale. Driller's log.

B-12 Village of Troy test hole 4, Layne-Western Co., 1953. Approx. 100 feet N, 2910 feet W of SE corner sec. 20, T. 3 N., R. 8 W., Madison Co. Elev. 432 feet. Total depth 88 feet, finished in shale. Driller's log.

B-13 Village of Troy test hole 1, Layne-Western Co., 1953. Approx. 100 feet N, 1860 feet W of SE corner sec. 20, T. 3 N., R. 8 W., Madison Co. Elev. 437 feet. Total depth 48 feet, finished in shale. Driller's log.

C–1 City of St. Louis River Front Project D. H. 116, 1951. 5700 feet S of 38° 37′ 30″ N, 5300 feet E of 90° 12′ 30″ W, St. Louis Co. Elev. 412 feet. Total depth 53.5 feet, finished in sand and gravel. Engineer's field log.

C–2 Corps of Engineers test hole W–77, 1952–53. 8400 feet N of 38° 35′ N, 3600 feet W of 90° 10′ W, T. 2 N., R. 10 W., St. Clair Co. Elev. 415 feet. Total depth 127 feet, bit refusal. Engineer's field log.

C–3 American Zinc Co., Monsanto, well 6, Harold L. Watson Drilling Co., 1940. 6900 feet N of 38° 35′ N, 750 feet W of 90° 10′ W, T. 2 N., R. 10 W., St. Clair Co. Elev. 405 feet. Total depth 107 feet, finished in soapstone. Driller's log.

C–4 Monsanto Chemical Co. test hole 4, Layne-Western Co., 1948. 5100 feet N of 38° 35′ N, 250 feet W of 90° 10′ W, T. 2 N., R. 10 W., St. Clair Co. Elev. 411 feet. Total depth 110 feet, finished on rock. Driller's log.

C–5 Socony-Vacuum Oil Co. well, Layne-Western Co., 1952. 2400 feet E of 90° 10′ W, 4400 feet N of 38° 35′ N, T. 2 N., R. 10 W., St. Clair Co. Elev. 410 feet. Total depth 117½ feet, finished in gravel and sand. Sample set 22655, studied by P. M. Busch.

C–6 Key Co. well, East St. Louis, Harold L. Watson Drilling Co., 1943. 6200 feet N of 38° 35′ N, 4700 feet W of 90° 07′ 30″ W. Total depth 117 feet, finished in sand and gravel. Driller's log.

C–7 Aluminum Ore Co. well, East St. Louis, Harold L. Watson Drilling Co., 1940. 4100 feet N of 38° 35′ N, 90° 07′ 30″ W, T. 2 N., R. 9 W., St. Clair Co. Elev. 417 feet. Total depth 121 feet, finished in fine sand and mud. Driller's log.

C–8 Illinois State Water Survey well 1, Layne-Western Co., 1951. 1800 feet S, 800 feet E of NW corner sec. 26, T. 2 N., R. 9 W., St. Clair Co. Elev. 422 feet. Total depth 81 feet, finished in sand. Sample set 21485, studied by W. H. Bierschenk.

C–9 Drive-in Theater well, French Village, Harold L. Watson Drilling Co., 1941. 450 feet W of SE corner sec. 23, T. 2 N., R. 9 W., St. Clair Co. Elev. 433 feet. Total depth 82½ feet, finished at shale. Driller's log.

D–1 Anheuser-Busch Co. test hole 1, Ranney Well Co. 2600 feet N of 38° 35′ N, 800 feet E of 90° 12′ 30″ W, St. Louis Co. Elev. 417 feet. Total depth 73 feet, finished on rock. Driller's log.

D–2 Alton and Southern Railroad well 2, Fox Terminal, Harold L. Watson Drilling Co., 1950. 100 feet S of 38° 35′ N, 1100 feet E of 90° 12′ 30″ W, T. 2 N., R. 10 W., St. Clair Co. Elev. 410 feet. Total depth 104 feet, finished in sand. Driller's log.

D–3 Corps of Engineers test hole W–95, 1952–53. 3400 feet S of 38° 35′ N, 1900 feet E of 90° 12′ 30″ W, T. 2 N., R. 10 W., St. Clair Co. Elev. 396 feet. Total depth 82 feet, finished in gravelly sand. Engineer's field log.

D–4 Corps of Engineers seepage well 2, Cahokia, 1952–53. 7250 feet N of 38° 32′ 30″ N, 90° 12′ 30″ W, T. 1 N., R. 10 W., St. Clair Co. Elev. 406 feet. Total depth 108 feet, finished on bedrock. Engineer's field log.

D–5 Corps of Engineers well W24B, Prairie du Pont, 1952–53. 5000 feet N of 38° 32′ 30″ N, 600 feet E of 90° 12′ 30″ W, T. 1 N., R. 10 W., St. Clair Co. Elev. 413 feet. Total depth 117 feet, bit refusal. Engineer's field log.

D–6 Corps of Engineers test hole DH-6-S, 1952. 3600 feet S, 1200 feet W of NE corner sec. 10, T. 1 N., R. 10 W., St. Clair Co. Elev. 416 feet. Total depth 84½ feet, bit refusal. Engineer's field log.

D–7 Corps of Engineers test hole DH, 1950–54. 2300 feet N, of 38° 32′ 30″ N, 1650 feet E of 90° 10′ W, T. 1 N., R. 10 W., St. Clair Co. Elev. 408 feet. Total depth 116 feet, bit refusal.

E–1 Tarlton and Sklar–Dyroff well 1–A, 1943. 1070 feet N, 820 feet W of SE corner sec. 28, T. 1 N., R. 10 W., St. Clair Co. Elev. 403 feet. Total depth 1800 feet, finished in Gasconade dolomite. Sample study 9318, studied by D. Speziale.

E–2 Lockwood–Dyroff well 1, 1924. 150 feet S of NW corner NE ¼ sec. 26, T. 1 N., R. 10 W., St. Clair Co. Elev. 590 feet. Total depth 2904 feet, finished in Potosi dolomite. Sample study 423, studied by F. E. Tippie.

E–3 Sewell–Bayless–Sparks well 1, 1931. SW ¼ NE ¼ SW ¼ sec. 2, T. 1 N., R. 10 W., St. Clair Co. Elev. 410.5 feet. Total depth 2002 feet, finished in Jefferson City dolomite. Sample study 1001, studied by Margaret Blair.

E–4 Monk's Mound well. Center NW ¼ NW ¼ NE ¼ sec. 2, T. 2 N., R. 9 W., St. Clair Co. Elev. 437 feet. Samples studied by J. A. Udden.

E–5 Commonwealth Steel Co. well. NW ¼ SW ¼ sec. 24, T. 3 N., R. 10 W., Madison Co. Elev. 423 feet. Total depth 2085 feet, finished in Jefferson City dolomite. Sample study 226, studied by A. Thurston.

E–6 Kesl–Kusmanoff well 1, 1947. 660 feet from N line. 330 feet from W line, SW ¼ SE ¼ sec. 12, T. 3 N., R. 9 W., Madison County. Elev. 410.6 feet. Total depth 1687 feet, finished in Kimmswick limestone. Sample study 17178, studied by M. P. Meyer and Heinz Lowenstam.

E–7 Penn–Illinois–Poag well 1, 1938. 2400 feet from S line, 3630 feet from E line, sec. 12, T. 9 N., R. 9 W., Madison Co. Elev. 424.6 feet. Total depth 2093 feet, finished in St. Peter sandstone. Sample study 8582, studied by T. C. Buschbach.

E–8 Lindberg Park well, 1932. 1830 feet from N line, 2320 feet from W line, sec. 8, T. 5 N., R. 9 W., Madison Co. Elev. 446.9 feet. Total depth 1200 feet, finished in Maquoketa shale. Sample study 935, studied by L. E. Workman.

F–1 Bethalto city well 3, Thorpe Concrete Well Co., 1951. 2200 feet N, 1200 feet W, SE corner sec. 22, T. 5 N., R. 9 W., Madison Co. Elev. 437 ± feet. Total depth 95 feet, finished in coarse sand, gravelly. Driller's log and sieve analysis.

F–2 Wood River city well 1, Thorpe Concrete Well Co., 1930. 860 feet S, 300 feet E, NW corner sec. 26, T. 5 N., R. 9 W., Madison Co. Elev. 446.7 feet. Total depth 109 feet, finished in pink sand. Sample study 1056, studied by L. E. Workman.

F-3 Shell Oil Co. well 59, Thorpe Concrete Well Co., 1952. NE ¼ SE ¼ SW ¼ sec. 35, T. 5 N., R. 9 W., Madison Co. Elev. 442 feet. Total depth 110 feet, finished in fine sand and small gravel. Samples studied by R. E. Bergstrom and T. R. Walker.

F-4 Shell Oil Co. well 61, Thorpe Concrete Well Co., 1952. NW ¼ SW ¼ SE ¼ sec. 35, T. 5 N., R. 9 W., Madison Co. Elev. 442 feet. Total depth 113 feet, finished in sand and gravel. Sieve analysis.

F-5 Sinclair Oil Co. well 1, Harold L. Watson Drilling Co., 1952. 1750 feet E, 460 feet N, SW corner sec. 34, T. 5 N., R. 9 W., Madison Co. Elev. 431 feet. Total depth 126 feet, finished in medium sand and gravel. Sample study 23403, studied by R. E. Bergstrom. Sieve analysis.

F-6 Union Starch and Refining Co. well, Harold L. Watson Drilling Co., 1952. 1000 feet N, 2800 feet E, SW corner sec. 13, T. 3 N., R. 10 W., Madison Co. Elev. 422 feet. Total depth 115 feet, finished in medium sand. Sample study 23406, studied by R. E. Bergstrom. Sieve analysis.

F-7 Collinsville city well 8, Layne-Western Co., 1951. SE ¼ SE ¼ sec. 31, T. 3 N., R. 8 W., Madison Co. Elev. 424 feet. Total depth 98 feet, finished in shale. Samples studied by W. H. Bierschenk. Sieve analysis.

F-8 Hunter Packing Co. well, Harold L. Watson Drilling Co., 1948. SE ¼ NW ¼ sec. 7, T. 2 N., R. 10 W., St. Clair Co. Elev. 418 feet. Total depth 115½ feet, finished in sand and gravel. Samples studied by R. E. Bergstrom. Sieve analysis.

F-9 Monsanto Chemical Co. well Z-12, Ranney Well Co., 1952. SE ¼ SE ¼ SE ¼ sec. 22, T. 2 N., R. 10 W., St. Clair Co. Elev. 400 feet. Total depth 97 feet, stopped on rock. Sample study 23443, studied by J. W. Baxter. Sieve analysis.

F-10 Cargill Co., Fox Terminal Elevator well, Harold L. Watson Drilling Co., 1952. NE ¼ SE ¼ NE ¼ sec. 33, T. 2 N., R. 10 W., St. Clair Co. Elev. 410 feet. Total depth 110 feet, finished in medium sand. Sample study 23404, studied by F. B. Titus. Sieve analysis.

APPENDIX 2

SIEVE ANALYSES OF LOWER PART OF VALLEY FILL, AMERICAN BOTTOMS

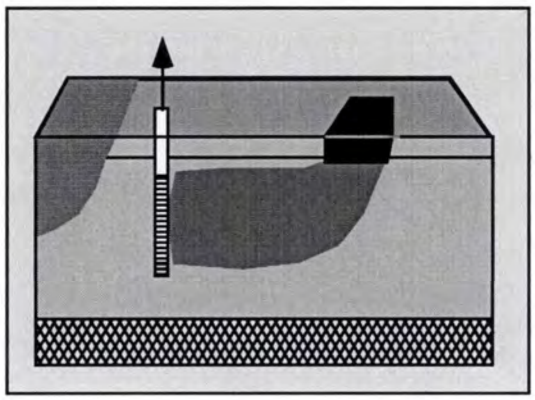
Sample	Percent by weight retained on screen										
	Mesh										
Well number; depth	4	9	16	24	32	42	60	80	115	170	Pan (silt and clay)
F–1											
65–70	—	—	—	1.2	3.7	7.	22.5	43.7	16.2	5.	.5
75–80	—	—	1.3	1.3	2.6	2.6	22.4	55.4	10.5	2.6	1.
85–90	—	6.6	22.8	27.6	10.4	1.9	13.3	13.3	2.8	.9	—
90–93	—	4.8	22.5	30.6	22.5	4.8	4.8	8.	1.6	—	—
F–2											
76.6– 93.1	—	17.	26.1	19.3	21.5	6.8	4.5	3.4	1.1	—	—
93.1–100.1	—	—	—	.9	4.8	29.8	50.9	10.5	1.9	.9	—
100.1–108.1	—	—	—	—	5.1	23.3	48.	19.4	2.5	1.2	—
F–3											
69– 81	—	—	.5	1.	7.9	24.4	44.5	15.3	4.6	1.	.2
81– 96	1.7	3.7	16.8	24.1	30.9	15.5	4.7	1.2	.7	.2	—
96–103	1.5	2.7	4.5	8.8	25.9	25.6	19.8	8.5	1.7	.5	—
103–110	1.2	1.2	1.9	2.6	24.3	42.4	15.1	7.5	2.4	.7	.2
F–4											
76– 84	1.3	1.8	5.9	6.6	20.9	33.8	15.2	8.2	3.6	1.6	.4
84– 97	25.5	14.	13.8	7.9	12.5	11.8	10.8	2.3	.8	.1	—
97–103	—	.9	2.4	2.4	9.8	49.6	26.5	6.3	.9	.4	.2
103–113	4.7	1.4	2.3	3.8	38.6	30.5	10.9	5.2	1.4	.4	.2
F–5											
70– 75	2.9	9.7	20.	16.2	25.4	17.3	7.8	1.2	.2	—	—
80– 85	7.9	11.	10.1	8.6	25.4	22.4	7.9	6.2	.2	—	—
90– 95	—	.9	2.2	2.9	11.7	41.	31.2	7.6	1.7	.2	.2
100–105	11.7	11.2	14.4	10.8	24.2	13.9	9.1	3.1	.2	—	—
F–6											
60– 70	13.4	16.5	14.6	12.8	14.7	10.5	8.2	3.7	3.5	1.5	.4
80– 88	.5	1.6	4.9	3.8	6.9	8.3	15.9	23.9	16.8	12.1	4.9
95–100	10.7	20.6	30.5	11.3	11.1	6.1	4.4	2.8	1.6	.2	.4
105–110	12.4	6.8	9.0	5.6	22.0	27.7	12.4	2.3	.6	.6	.6
B–9											
60– 65	8.5	12.8	12.8	9.4	15.6	15.6	12.8	7.1	2.3	.9	.9
70– 75	5.9	8.9	13.1	16.5	21.2	18.6	9.7	4.2	.8	.4	.4
80– 85	.4	2.2	10.9	14.5	35.9	22.7	8.2	3.2	.9	.4	.4
90– 95	—	7.7	18.8	24.3	24.8	14.9	6.	2.2	.5	.2	.2
100–105	—	.9	27.7	27.2	25.3	9.5	5.7	2.4	.4	.2	.4
F–7											
75– 80	28.0	24.3	26.8	7.3	6.0	3.6	1.2	.6	.6	.6	.6
85– 90	15.9	11.6	31.	23.5	12.6	3.3	.8	.4	.4	—	—
95–100	1.2	2.4	17.	30.4	31.7	12.8	2.4	1.2	.6	—	—
F–8											
75– 80	5.8	7.1	8.1	8.1	17.8	22.4	15.8	9.9	2.5	1.5	.5
85– 90	11.9	13.5	10.4	17.1	32.8	10.4	2.3	1.	.2	—	—
95–100	3.8	5.3	18.	20.7	39.7	9.5	1.6	.4	—	—	—
F–9											
68–79	5.3	2.3	6.5	8.9	20.1	29.	16.3	5.9	3.2	1.4	.5
79–86	22.5	8.5	8.5	7.9	16.8	19.4	11.4	2.8	.8	.5	.2
86–90	1.1	3.0	13.6	28.2	35.	14.8	2.8	.2	.4	—	.4
90–96	10.2	7.0	17.2	17.2	30.6	13.1	2.9	.4	.4	—	.2
F–10											
70– 75	2.1	8.3	11.9	7.6	16.2	23.4	17.4	8.8	2.6	.9	.2
80– 85	1.4	1.7	2.4	2.9	11.8	12.5	24.8	28.8	6.1	4.6	2.4
90– 95	—	.2	.7	.7	19.	54.3	17.2	4.9	1.9	.4	.2
95–100	—	—	.7	.4	19.8	55.2	19.5	3.4	.4	.2	—

Illinois State Geological Survey, Report of Investigations 191
44 p., 4 pls., 6 figs., 2 tables, 1956

MASS CONTAINMENT STUDY - JAN. 2002

Site R
W.G. Krummrich Plant

Sauget, Illinois



PRELIMINARY

Submitted to
Solutia Inc.

January 24, 2002

Groundwater Services, Inc.
2211 Norfolk, Suite 1000, Houston, Texas 77098

GROUNDWATER SERVICES, INC.

EXECUTIVE SUMMARY

Site R is located in the American Bottoms area on the east bank of the Mississippi River and west of the W.G. Krummrich Plant. In this report, Site R refers to a capped area approximately 2000 ft wide (perpendicular to groundwater flow) and 500 long (parallel to groundwater flow). Below Site R, affected groundwater extends from close to the water table to bedrock (typically from 30 ft to 140 ft below ground surface).

The objective of this study was to evaluate the efficiency of contaminant mass capture by a groundwater recovery system operating at various system flow rates. This objective included estimation of the mass flux of contaminants from Site R to the Mississippi River via groundwater transport and evaluation of the effect of groundwater pumping on the transport of contaminants to the river.

A MODFLOW/MT3D model of the site (Figures 1 and 2) was used to estimate the mass flux and subsequent reduction in mass flux achieved by the groundwater recovery system. For this study, a two-well pumping system was evaluated in the model.

Results

Based on several model runs with different total system flowrates, the following mass flux capture percentages were derived:

- 200 gpm total system flowrate gpm captures 35% of mass flux to river
- 350 gpm total system flowrate gpm captures 60% of mass flux to river
- 500 gpm total system flowrate gpm captures 75% of mass flux to river
- 650 gpm total system flowrate gpm captures 85% of mass flux to river

A graph indicating mass flux captured vs. total system flowrate is shown in Figure 3.

Note that these mass flux capture results are based on model simulations with inherent uncertainty in both the model and model input data. More accurate mass flux capture data would be obtained by field trials of the actual recovery system.

INTRODUCTION

As requested by Solutia Inc. (Solutia), Groundwater Services, Inc. (GSI), has completed a study of mass containment options for affected groundwater associated with Site R near the W.G. Krummrich Plant in Sauget, Illinois. This report summarizes the approach and results of the study.

PROJECT BACKGROUND

Site R is located in an area referred to as the American Bottoms on the east bank of the Mississippi River directly downgradient of the W.G. Krummrich Plant. The geology of the area is described as consisting of unconsolidated valley fill deposits (Cahokia Alluvium) overlying glacial outwash material (Henry Formation). In general, the permeability of the unconsolidated material increases with depth with the outwash material being comprised of medium- to coarse-grained sand and gravel. The hydrogeologic conceptual model (Figure 1, GSI, 2001) divides the unconsolidated water-bearing unit into three horizons: the shallow horizon (extending 400 to 380 ft MSL), the middle horizon (extending from 380 to 350 ft MSL) and the deep horizon (extending from 350 ft MSL to bedrock, or about 290 ft MSL at Site R).

In this report, Site R refers to a capped area approximately 2000 ft wide (perpendicular to groundwater flow) and 500 ft long (parallel to groundwater flow).

Representative constituents associated with Site R include volatile organic constituents (VOCs) such as benzene, chlorobenzene, acetone, and 1,2-dichloroethane and semi-volatile organic constituents (SVOCs) such as phenol, 2-chloroaniline, and 2-nitrochlorobenzene. Site constituents are found from the water table to bedrock in all three horizons.

The objective of this study was to evaluate the efficiency of contaminant mass capture by a groundwater recovery system operating at different total system flowrates

A numerical groundwater flow model, MODFLOW, and a mass transport model, MT3D, were used to evaluate these alternatives (Waterloo Hydrogeologic, 1999).

MODEL DESCRIPTION

The MODFLOW groundwater flow model, developed by the U.S. Geological Survey (McDonald and Harbaugh, 1988) was used to simulate the movement of groundwater under baseline conditions and for various pumping scenarios. The MT3D mass transport model (Waterloo Hydrogeologic, 1999) was used in conjunction with MODFLOW to evaluate the movement of dissolved constituents migrating in the groundwater.

Key MODFLOW Model Attributes, Assumptions, and Input Parameters

The MODFLOW model used in this report is described in the "Discharge Control Study" (Groundwater Services, Inc., Nov. 29, 2001). Details of the model attributes, assumptions, and input data are described in that report, and in subsequent response to comments.

Key MT3D Model Attributes, Assumptions, and Input Parameters

MT3D is a contaminant transport model that simulates the transport of dissolved constituents under the influence of advection (bulk groundwater flow), dispersion (spreading of constituent paths due to diffusion and preferential flowpaths), sorption (the adsorption of constituents to the aquifer media), and degradation (the destruction of constituents by chemical or biological processes). MT3D runs on top of the MODFLOW model using the same model grid (Figure 1). For the MT3D model:

- Adsorption and biodegradation were ignored in the simulations performed for this project to yield a conservative mass capture simulation. Dispersion was set a relatively low value to focus on this advection-dominated process and to minimize computational problems.

- Constant concentration sources were assumed to exist in the upper, middle, and deep aquifers. Source strengths were determined using the geometric mean of concentrations obtained within the highest concentration contours of the SVOC and VOC plume maps, respectively, developed by Roux Associates, Inc. (2000).

MT3D Modeling Approach

To establish representative starting concentrations, MT3D was run for 30 years, and the resulting concentrations caused by the source terms were compared to the concentrations observed in monitoring wells at the site. By adjusting the source locations and strengths the MT3D model's predicted concentrations were within reasonable agreement with observed concentrations at the site. This 30-year concentration distribution was then used as the initial condition for all subsequent mass transport modeling (Figure 2).

One project objective was to determine dissolved constituent mass flux discharge to the river under various pumping rates. For this calculation, the baseline quantity of groundwater flowing into the river and the concentration of dissolved constituents in the groundwater discharged to the river was needed. These quantities were calculated using the ZoneBudget feature of MODFLOW in conjunction with mass transport simulations using MT3D.

ZoneBudget is a water balance component within Visual MODFLOW that calculates the exchange of groundwater between adjacent user-established zones. The calculation accounts for inflow into a zone from all sources and outflow through model edges and internal sinks. To calculate the transfer of water from the area under site R to the river, numerous separate zones were defined representing the aquifer adjacent to the river and the river itself. Separate river and aquifer zones were established for each horizon since initial constituent concentration differed between layers. The quantity of water flowing from each layer into the river zone was calculated by ZoneBudeget and the sum was used as water flow to the river.

Each horizon near Site R was divided into 10 zones. For each pumping regime, the mass lost to the river was calculated by the following procedure:

1. For each modeling scenario, MODFLOW, ZoneBudget and MT3D were run. The rate of groundwater discharge to the river from each aquifer zone reported by ZoneBudget was then used in the mass balance calculations.
2. The concentration in each aquifer zone that discharged to the river was estimated by placing a concentration observation well in each horizon zone. This concentration represented the dissolved constituent concentration discharged to the river from each zone. The concentrations were recorded by MT3D at periodic intervals for use in the mass balance calculations.

3. The total mass discharged to the river over the modeling period was calculated as the sum of the products of the river discharge and concentrations (after a five year simulation) in each zone as follows:

$$M_R = \sum_{1}^{\text{number of zones}} Q_i C_i$$

where Q_i = discharge rate of groundwater from zone *i* into the river
C_i = final constituent concentration in zone *i*
M_R = mass discharged to river

Modeling Results

A two well system with fully penetrating wells was used for a series of model simulations with different pumping rates from the pumping system. The mass flux with the pumping system in place was determined using the model and the equation described above, and then compared to the no-pumping scenario. The model simulations indicated that higher flows captured more mass (Figure 3):

Total System Flowrate (gpm)	Percentage of Mass Flux To River Captured by Recovery System (%)
0	0%
250	45%
500	76%
1000	96%
1500	100%

These results were used to construct the curve shown on Figure 3. By interpolating values on this curve, the following results were derived:

- 200 gpm total system flowrate gpm captures 35% of mass flux to river
- 350 gpm total system flowrate gpm captures 60% of mass flux to river
- 500 gpm total system flowrate gpm captures 75% of mass flux to river
- 650 gpm total system flowrate gpm captures 85% of mass flux to river

Note that these mass flux capture results are based on model simulations with inherent uncertainty in both the model and model input data. More accurate mass flux capture data would be obtained by field trials of the actual recovery system.

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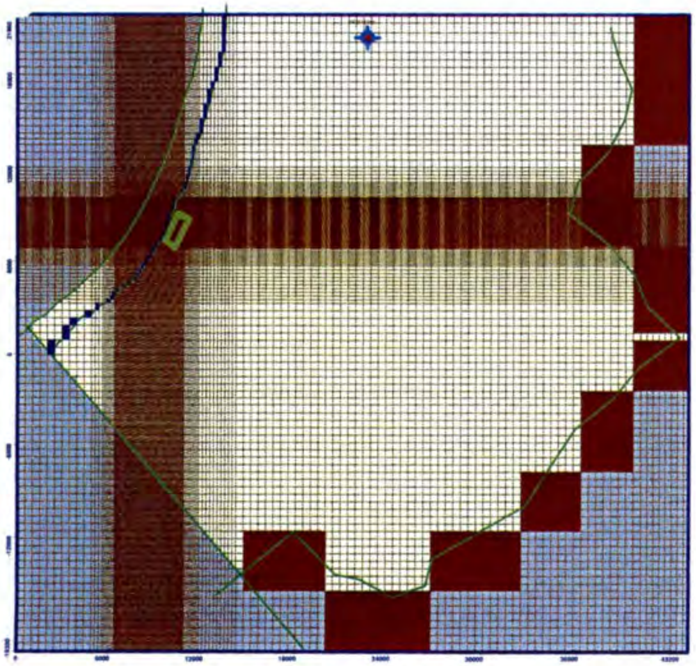
GSI Job No. G-2561
January 24, 2002

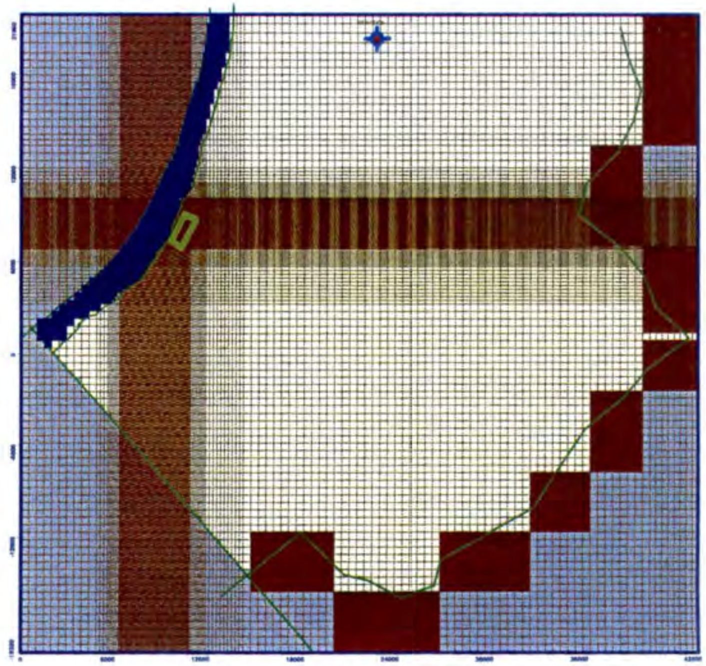
GROUNDWATER SERVICES, INC.

Mass Containment Study
Site R - W. G. Krummrich Plant
Sauget, Illinois

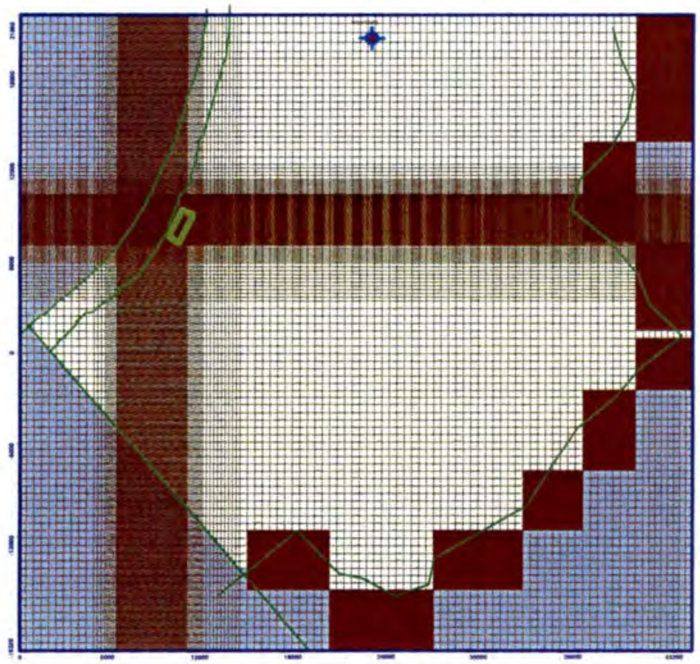
Figures



Layer 1: Shallow Horizon



Layer 2: Middle Horizon



Layer 3: Deep Horizon

N

SCALE (ft)

0 6000 12,000

PRELIMINARY

LEGEND

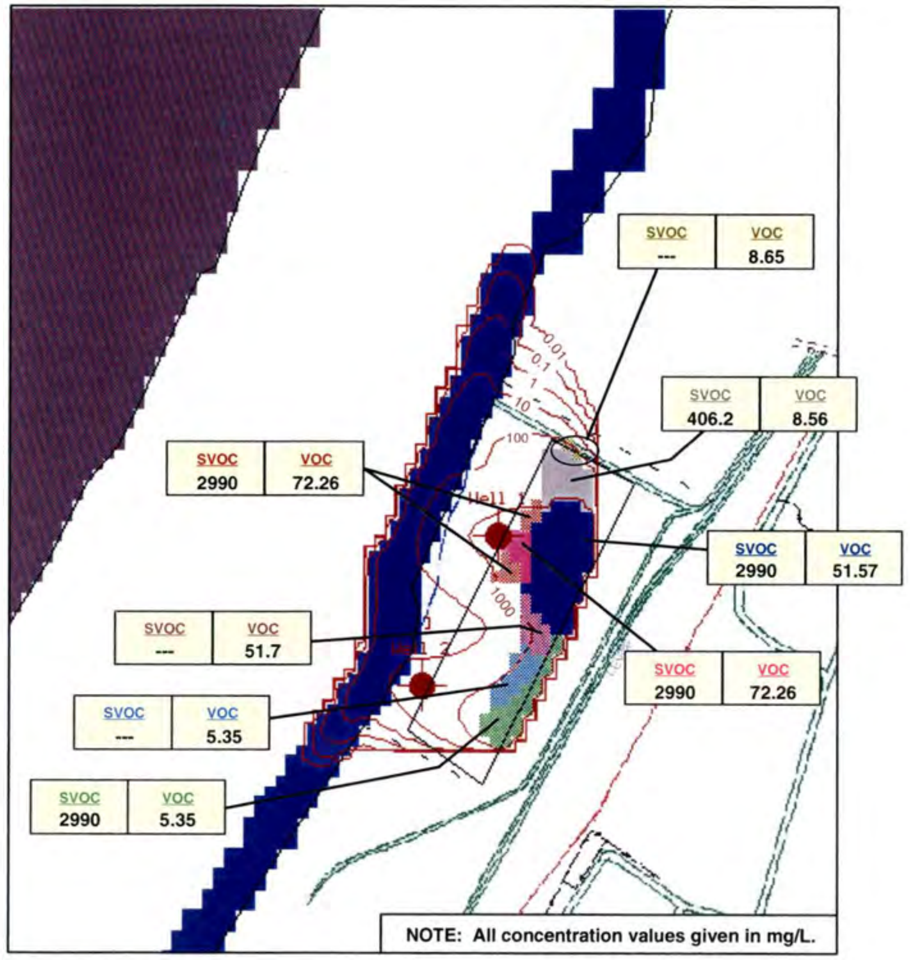
- Regional pumping center for highway dewatering
- River cells in MODFLOW / MT3D model
- Approximate area of Site R
- Constant head cells
- Inactive cells

GROUNDWATER SERVICES, INC.

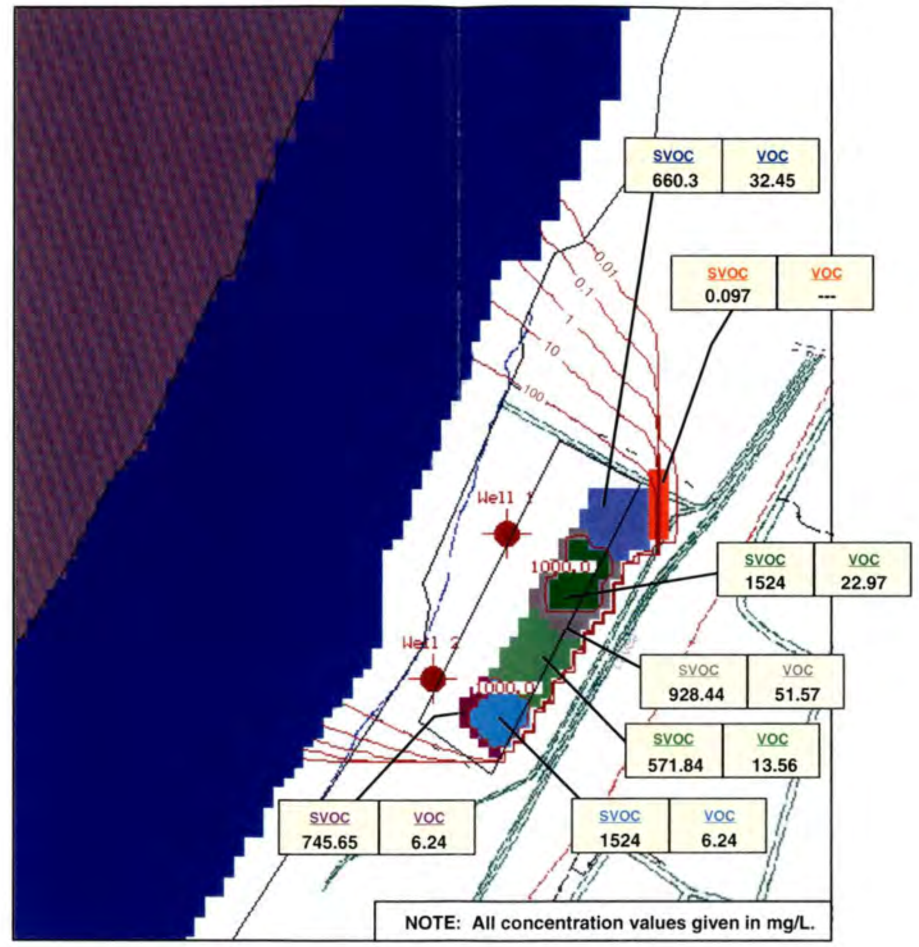
MODFLOW / MT3D MODEL CONFIGURATION MASS CONTAINMENT STUDY

Site R, W.G. Krummrich Plant, Sauget, Illinois
Solutia Inc.

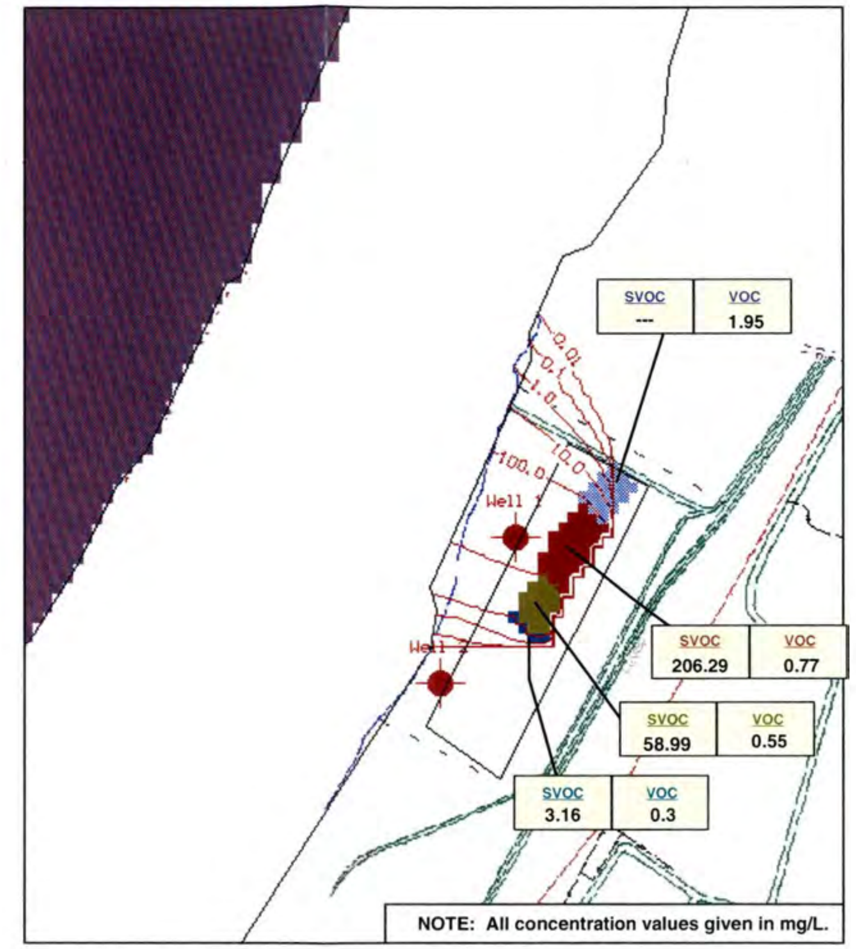
GSI Job No:	G-2561	Drawn By:	CRW
Issued:	1/24/02	Chk'd By:	SKF
Revised:	———	Appv'd By:	CJN
Scale:	As Shown		**FIGURE 1**



Layer 1: Upper Horizon (approximately 0 - 26 ft BGS)



Layer 2: Middle Horizon (approximately 26 - 77 ft BGS)



Layer 3: Deep Horizon (approximately > 77 ft BGS)

LEGEND

- 0.1 — Equal concentration line SVOC and VOC, mg/L
- Well
- Mississippi

NOTE:

1) Concentrations derived from 2000 - 2001 monitoring data and model calibration.

SCALE (ft)

0 600 1200

GROUNDWATER SERVICES, INC.

GSI Job No.	G-2561	Drawn By:	CRW
Issued:	1/23/02	Chk'd By:	SKF
Revised:	————	Aprv'd By:	CJN
Scale:	As Shown		**FIGURE 2**

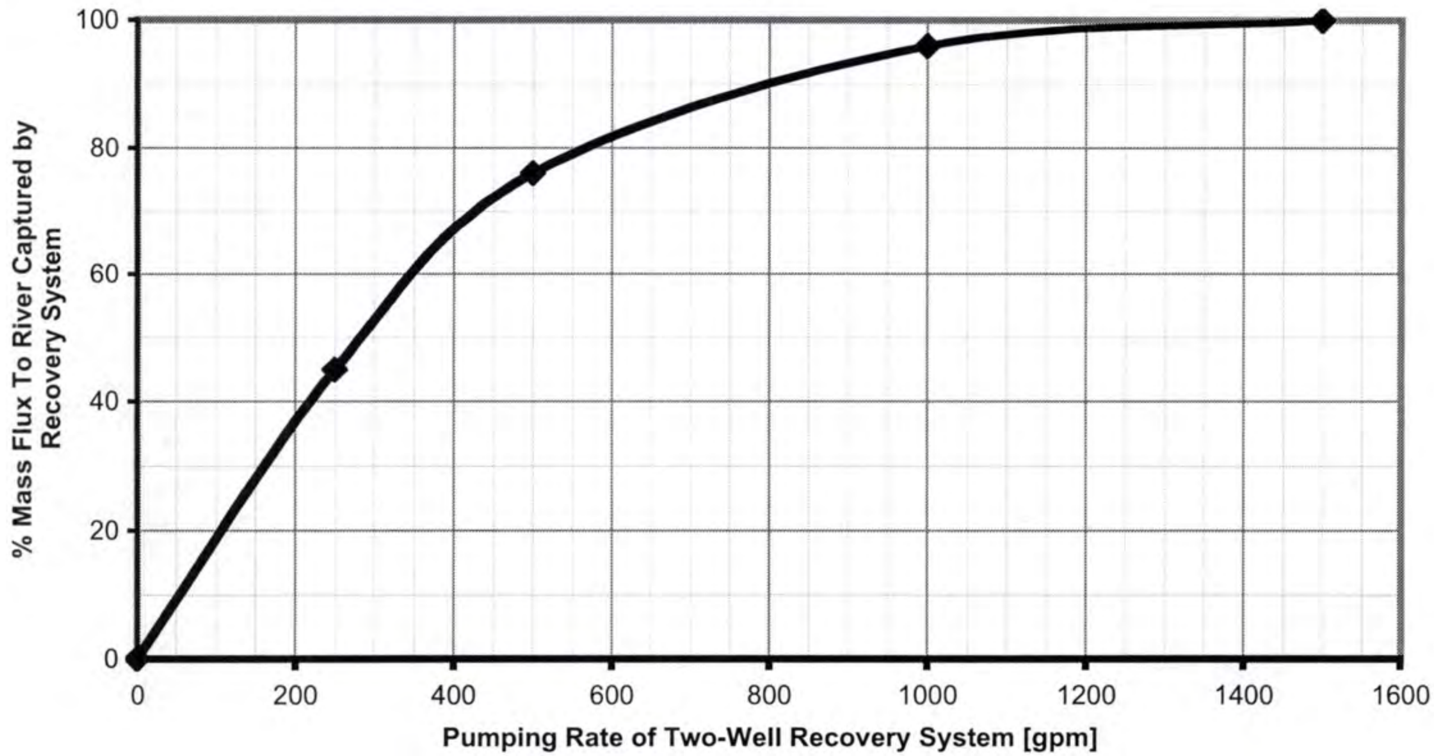
INITIAL CONSTITUENT CONCENTRATIONS USED IN MT3D MODEL MASS CONTAINMENT STUDY

Site R, W.G. Krummrich Plant, Sauget, Illinois
Solutia Inc.

GSI Job. No. G-2561
Issued: 1/24/02

Figure 3
ESTIMATED MASS CAPTURE VS. PUMPING RATE OF RECOVERY SYSTEM FROM MODFLOW/MT3D MODELING RESULTS

Site R, W.G. Krummrich Plant, Illinois
Solutia Inc.



Circular 180

Ground-Water Levels and Pumpage in the Metro-East Area, Illinois, 1986-1990

by Richard J. Schicht and Andrew G. Buck

Illinois State Water Survey
A Division of the Illinois Department of Natural Resources

1995

Circular 180

Ground-Water Levels and Pumpage in the Metro-East Area, Illinois, 1986-1990

by

Richard J. Schicht and Andrew G. Buck

Title: Ground-Water Levels and Pumpage in the Metro-East Area, Illinois, 1986-1990

Abstract: This report discusses ground-water levels and pumpage in the Metro-East area just south of Alton, Illinois, to Dupo, Illinois, and between the Mississippi River and the river bluffs from 1986-1990. Large quantities of ground water, primarily for industrial and municipal use, are withdrawn from wells penetrating a sand-and-gravel aquifer along the valley lowlands of the Mississippi River.

Ground-water pumpage declined from 62.8 million gallons per day (mgd) in 1986 to 58.7 mgd in 1990. Of the total 1990 pumpage, 76.2 percent (or 44.7 mgd) was industrial; 20.8 percent (or 12.2 mgd) was for public water supplies; 2.0 percent (or 1.2 mgd) was for irrigation; and 1.0 percent (or 0.6 mgd) was for domestic use. Pumpage in the Metro-East area is concentrated at five major pumping centers (Alton, Wood River, Roxana, National City, and Granite City) and four minor pumping centers (Poag, Glen Carbon, Collinsville, and Venice). Pumpage in the Sauget (Monsanto) area, once considered a minor pumping center (Kohlhase, 1987), was negligible in 1990 because of declining industrial use.

Ground-water levels throughout the entire area were stable but elevated during 1986 and 1987. Water levels declined from 1988 to 1989 and increased in 1990. Factors contributing to this pattern were above-normal precipitation, the Midwestern drought of 1988-1989, changes in river stages, and the response of water levels to annual pumpage changes.

Reference: Schicht, Richard J. and Buck, Andrew G. Ground-Water Levels and Pumpage in the Metro-East Area, Illinois, 1986-1990. Illinois State Water Survey, Champaign, Circular 180. 1995.

Indexing Terms: Metro-East area, ground water, public water supplies, industrial water supplies, water levels, water-level changes, pumping, ground-water withdrawals.

CONTENTS

Page

GROUND-WATER LEVELS AND PUMPAGE IN THE METRO-EAST AREA, ILLINOIS, 1986-1990

by Richard J. Schicht and Andrew G. Buck

ABSTRACT

This report discusses ground-water levels and pumpage in the Metro-East area just south of Alton, Illinois, to Dupo, Illinois, and between the Mississippi River and the river bluffs from 1986-1990. Large quantities of ground water, primarily for industrial and municipal use, are withdrawn from wells penetrating a sand-and-gravel aquifer along the valley lowlands of the Mississippi River.

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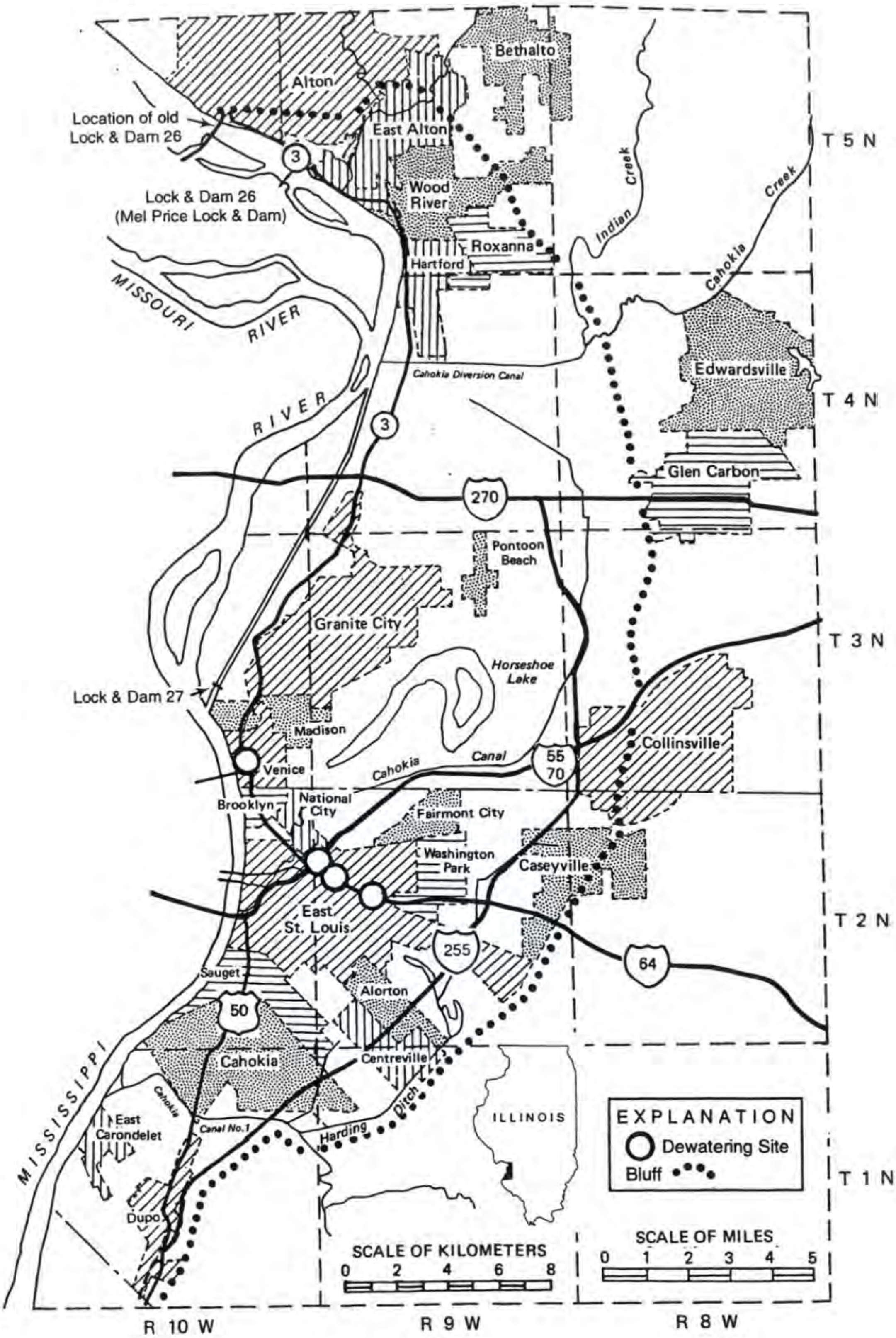
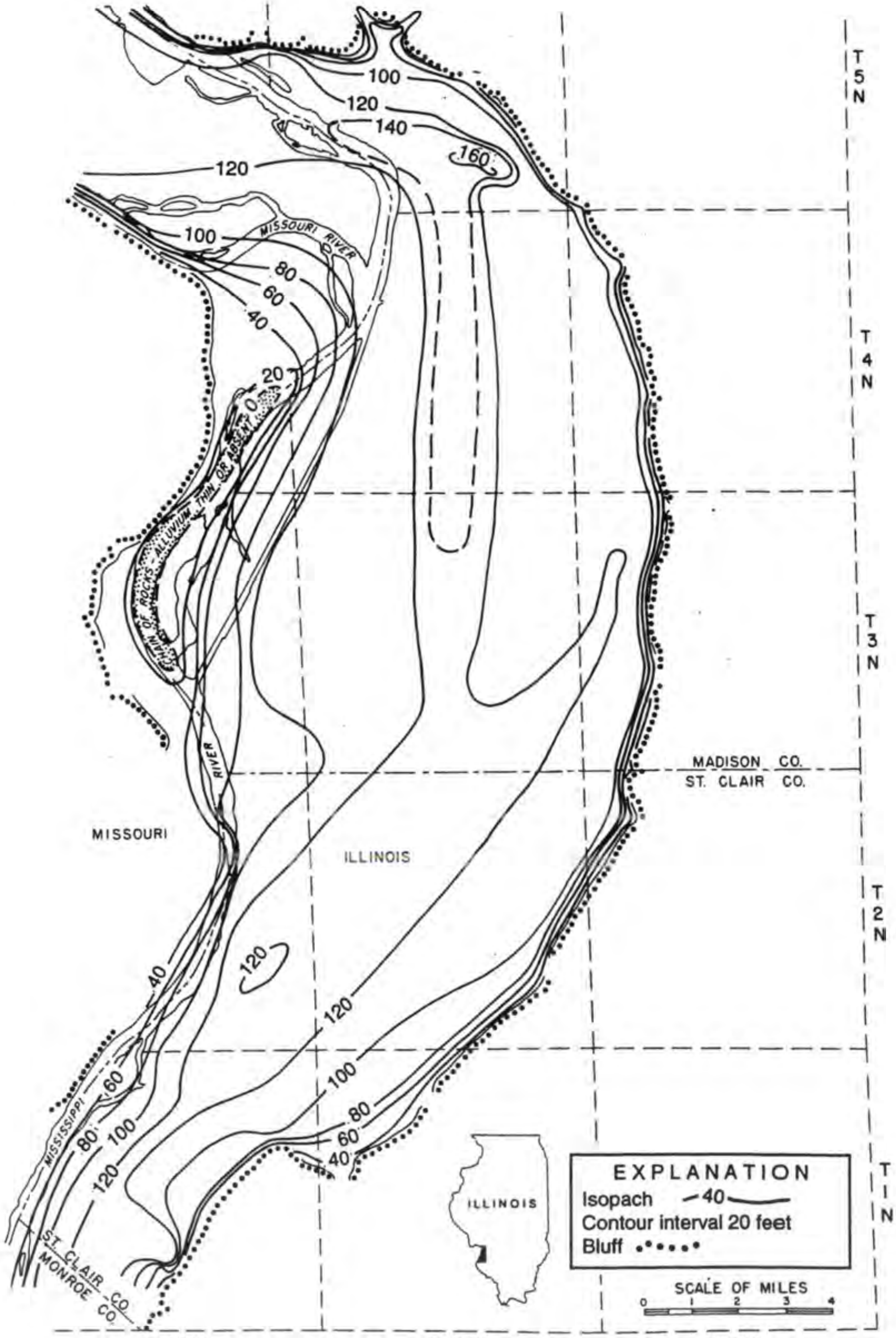


Figure 1. Location of the Metro-East area



(after Bergstrom & Walker, 1956)

Figure 2. Thickness of valley fill deposits

Pumpage, 1986-1990

Table 1 shows total pumpage, including all water use categories for the period 1986-1990. Total pumpage declined from 62.8 mgd in 1986 to 58.7 mgd in 1990. Distribution of 1990 pumpage is as follows: public supply systems (20.8 percent or 12.2 mgd), industrial pumpage (76.2 percent or 44.7 mgd), domestic pumpage (1.0 percent or 0.6 mgd), and irrigation pumpage (2.0 percent or 1.2 mgd).

Public Supplies. Municipal and institutional uses are included in public supplies. Pumpage for institutional use in the area has been negligible, however. Figure 4 shows the estimated pumpage for public supplies, which averaged 12.2 mgd for each year except 1988 when it was 13.3 mgd.

Pumpage of public supplies reflects seasonal variations to some extent. For example, municipal pumpage is generally 25 to 30 percent higher during the summer months than during the winter months because of lawn sprinkling, car washing, and other summer use of water.

Industrial Supplies. The major industrial users of ground water in the Metro-East area include oil refineries, chemical plants, ore refineries, meat packing plants, and steel plants. With its system of dewatering wells, IDOT is a major industrial user. Most industries do not meter their pumpage, and pumpage estimates are typically based on the number of hours the pump operated, on pump capacity, and in some cases on production capacity. Industrial pumpage generally is more uniform throughout the year than public pumpage unless large air-conditioning systems are used, the industry is seasonal, or a change in operation occurs as a result of strikes or vacation shutdowns. Industrial pumpage (figure 4) declined from 49.2 mgd in 1986 to 44.7 mgd in 1990.

Domestic Supplies. Estimates of domestic pumpage considered rural populations as reported by the U.S. Bureau of the Census and the per capita use of 84 gallons per day (gpd) used by Kohlhase (1987). On the basis of this per capita use, average domestic use in 1990 was estimated to be 600,000 gpd.

Irrigation Supplies. In 1989, a questionnaire was mailed to all known irrigators in the Metro-East area requesting information for 1988 on number of acres irrigated, type of crop irrigated, frequency of irrigation, and quantity of water applied. Based on the survey results, it was estimated that an average of about 0.7 mgd of ground water was withdrawn for irrigation during 1988. Respondents included 18 farmers who irrigated a total of 2000 acres. Estimated irrigation was 0.8 mgd in 1986 and 1989, 1.2 mgd in 1990, and less than 0.1 mgd in 1987, based on June-August rainfall measured at Belleville (table 2).

Table 1. Annual Pumpage (mgd), 1986-1990

Year	*Pumpage*
1986	62.8
1987	60.4
1988	61.6
1989	58.1
1990	58.7

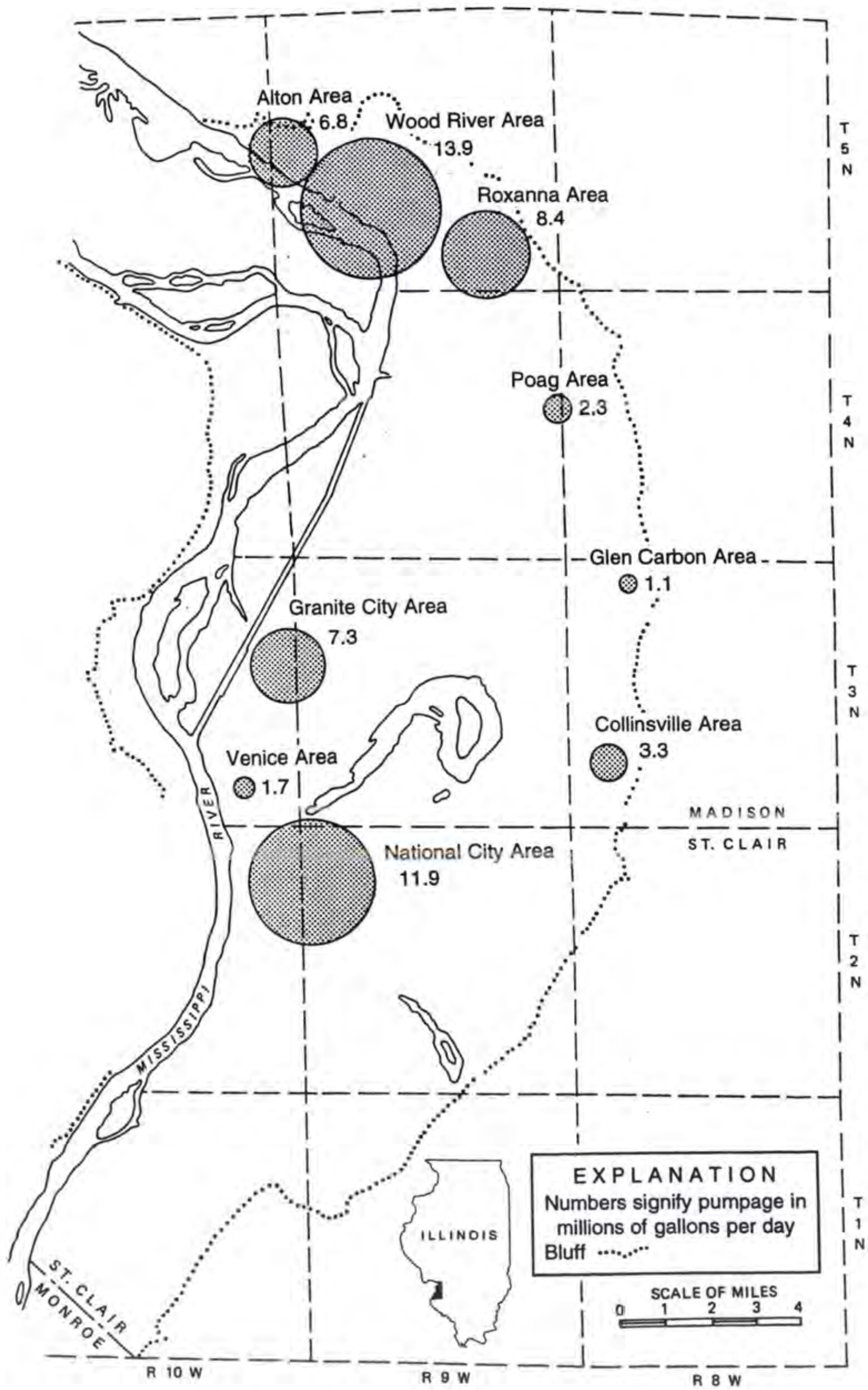


Figure 5. Distribution of estimated pumpage, 1990

Previous reports have included pumpage from the highway dewatering site at Venice in the total for National City. Sauget is no longer listed as a minor pumping center (Kohlhase, 1987), and pumpage there was negligible in 1990.

Figure 6 shows pumpage for 1981-1990 for each major pumping center. Ground-water withdrawals in the Alton area are primarily from wells owned by two industries and a municipality. During the 1986-1990 period pumpage at Alton varied from 6.7 mgd to 7.0 mgd, except during 1987 when pumpage was only 5.6 mgd because of reduced industrial activity.

The Wood River/Roxana area is the largest pumping center in the Metro-East area. Annual pumpage during 1986-1990 was fairly stable, varying from 22.3 mgd to 23.3 mgd. Pumpage in the Wood River/Roxana area is mainly for oil refineries and municipalities.

Ground-water pumpage in the Granite City area was about 10 mgd in 1986 and 1987. Pumpage declined to 7.4 mgd in 1988 and was 7.3 mgd in 1990. Steel production industries are the major ground-water users in the area.

Ground-water withdrawals in the National City area are mainly from wells at the interstate dewatering sites shown in figure 1 and at a paint pigment plant. Withdrawals for the meat packing industry, formerly large users, averaged only about 0.25 mgd in 1990. Since the goal of the dewatering sites is to maintain the ground-water elevations within the pumping centers at a relatively constant elevation, pumpage from wells at the sites fluctuates in response to changes in river stages, changes in recharge from precipitation, and changes in ground-water pumpage in the vicinity of the sites. Pumpage for the 1986-1990 period was highest in 1988 (12.8 mgd) and lowest (11.5 mgd) in 1989.

Figure 7 shows combined pumpage for the minor pumping centers. Except for the dewatering site at Venice, pumpage from these centers was mainly by municipalities. Pumpage for the period was highest (9.7 mgd) in 1989 and lowest (8.6 mgd) in 1990.

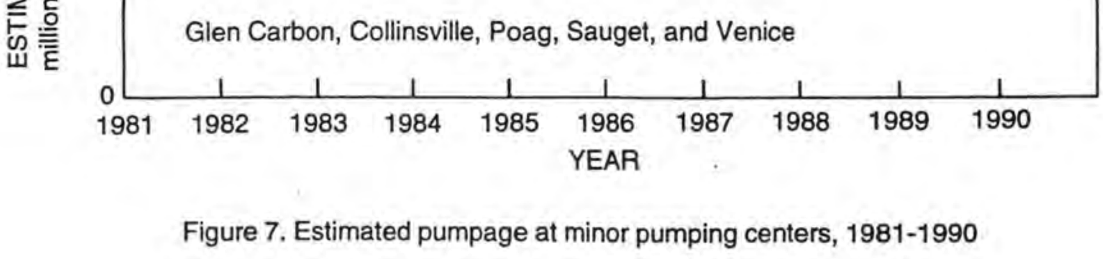


Figure 7. Estimated pumpage at minor pumping centers, 1981-1990

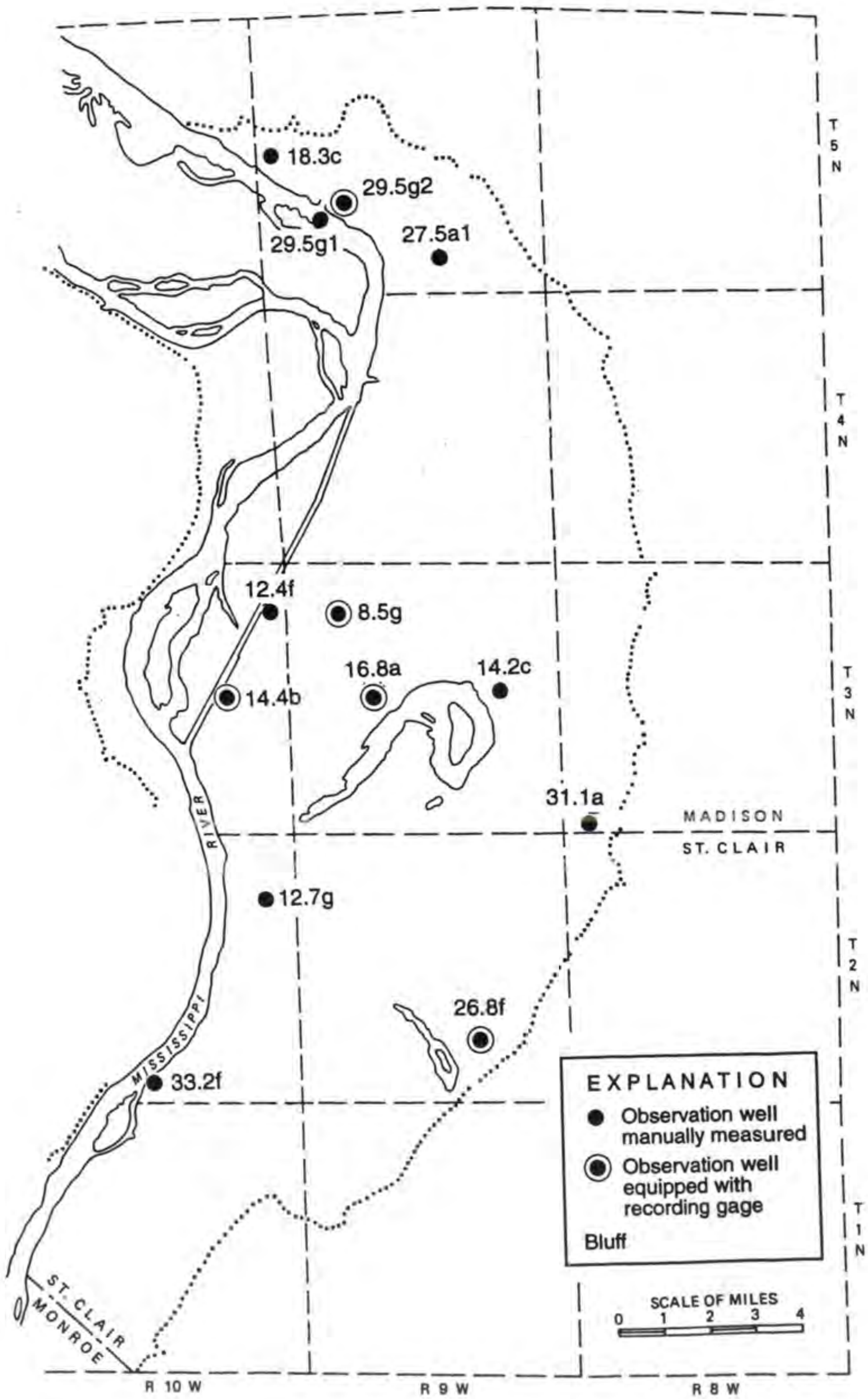


Figure 8. Locations of Illinois State Water Survey observation wells

Since 1900, ground-water levels have changed appreciably at the five major pumping centers. According to Schicht and Jones (1962), the greatest water-level declines for the period from 1900 to November 1961 occurred in major pumping: 50 feet in the Sauget area (formerly a major pumping center), 40 feet in the Wood River area, 20 feet in the Alton area, 15 feet in the National City area, and 10 feet in the Granite City area. Part of the declines, 2 to 12 feet, was attributed to the construction of levees and drainage ditches.

Reitz (1968) and Baker (1972) described the changes in ground-water levels from 1962-1971. Ground-water levels generally continued to decline through 1964, but began to rise about 1965 as the effects of decreased pumpage and above-average precipitation and river stages became noticeable.

Ground-water levels generally continued to rise for the period from 1972-1977 (Emmons, 1979). Decreases in pumpage caused ground-water levels to rise 2 feet in the Sauget and Wood River areas and 5 feet in National City. Little change was observed in the Alton and Granite City pumping centers. In Alton, a change of observation wells to a site nearer the center of pumpage obscured the rise in ground-water levels resulting from a decrease in pumpage. Erratic pumpage in the Granite City area produced small observed changes in ground-water levels.

During the period from 1978-1980 ground-water levels outside pumping centers showed little change (Collins and Richards, 1986). Trends established between 1971 and 1977 continued near pumping centers. Decreases in water levels in areas near the Mississippi River were generally due to low river stages. Decreases in water-level elevations of more than 5 feet in the Wood River area, however, were attributed to a change in the spatial distribution of pumpage. Ground-water levels in the Granite City area generally rose in proportion to decreased pumpage. Increased pumpage in the National City area expanded the area of declining ground-water levels near the river. Ground-water levels continued to recover in the Sauget area with reduced pumpage.

The trend in ground-water levels from 1981-1985 was for increasing water levels during 1981 and 1982, with apparent stabilization within an elevated range during 1983-1985 (Kohlhase, 1987). Above-normal precipitation and river stages from 1982-1985, coupled with the response of water levels to annual pumpage changes, were the main factors contributing to this trend in water levels. From 1981-1982, ground-water level increases of as much as 17 feet were observed in the National City and Alton areas, 8 feet to 16 feet in the Granite City region, 12 feet in the Wood River area, and 7 to 14 feet in areas near the bluff. Water levels stabilized at an elevated state after this trend of increasing water levels.

Figure 10 shows the mean monthly Mississippi River stages for the period from 1981-1990, and figure 11 shows the observed annual precipitation for the same period at Belleville (the raingage lies one mile south of Scott Air Force Base). Figure 12 shows hydrographs of selected wells for this period. A single line hydrograph represent water levels for wells at which the water level is measured monthly. A double line represents water levels for wells equipped with continuous recorders; the lines represent the observed monthly high and low ground-water levels.

The hydrographs show that these wells all share a similar fluctuation pattern from 1986-1990, differing only in magnitude of fluctuation. The general trend during this period was for stable water

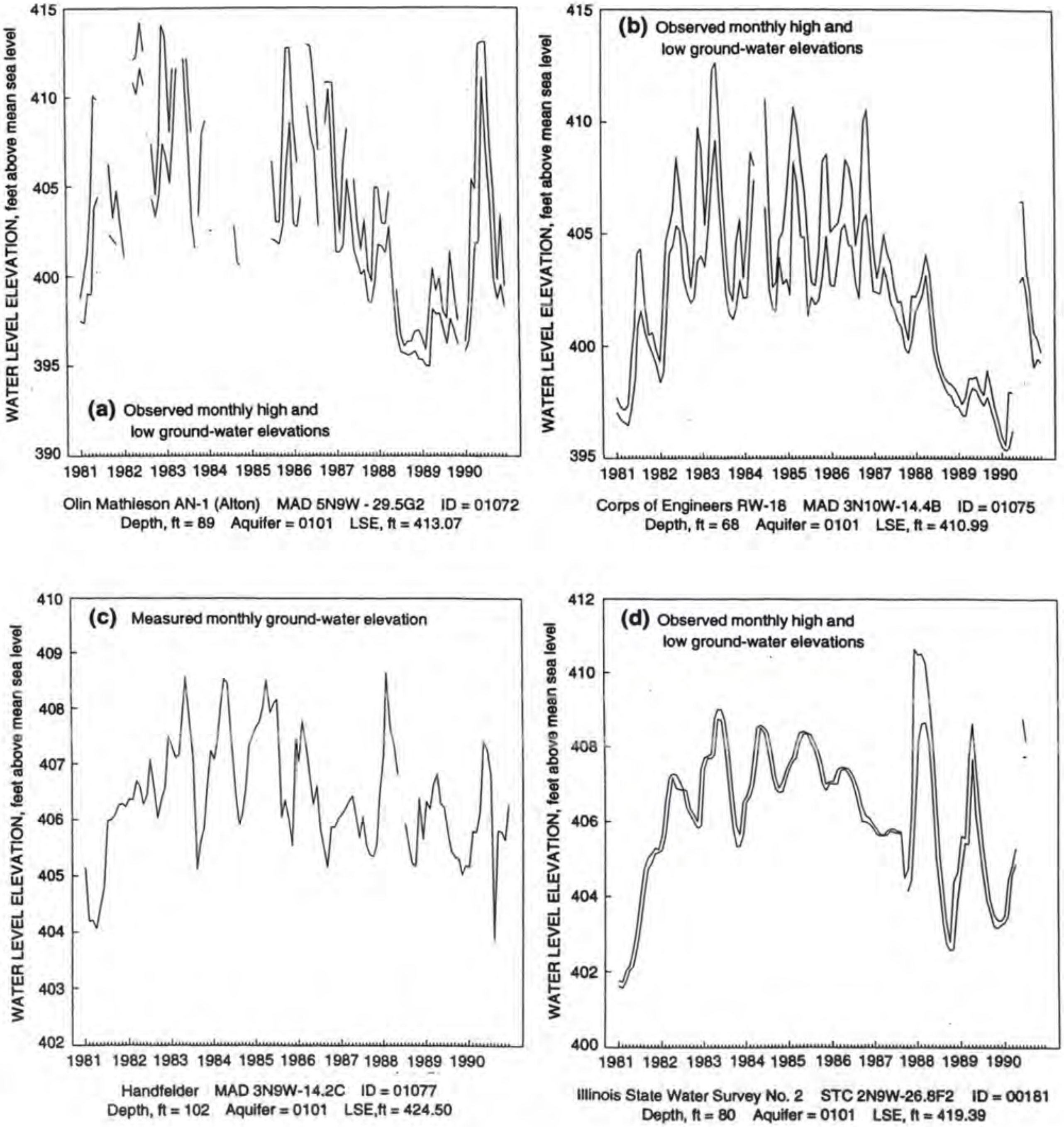


Figure 12. Hydrographs of eight selected wells, 1981-1990

correlated closely to ground-water fluctuations for the same time period. In relation to the 120-year mean river elevation, river stages during this time period had a below- and above-average pattern similar to the precipitation pattern.

From 1986-1990, ground-water levels in Well MAD5N9W-29.5g2 (figure 12a) and Well MAD3N10W-14.4b (figure 12b) generally reflect Mississippi River stages. Corresponding peaks in both ground-water hydrographs reflect high and low river stages. The effects of the drought of 1988-1989 are very evident (declining water levels) in both the mean monthly Mississippi River stage graph (figure 11) and in the hydrographs for both wells.

The magnitude of water-level change from 1986-1990 was controlled by each well's proximity to pumping centers and to the Mississippi River and other surface water bodies. Well MAD3N9W-14.2c (figure 12c) near the northeast end of Horseshoe Lake is a good example of a well that is not strongly affected by a pumping center and that has the stabilizing influence of Horseshoe Lake nearby and no drainageway in the immediate area. These conditions result in an annual fluctuation of water levels in this well of about 3 feet, more variation than in Well MAD3N9W-16.8a (figure 10a) discussed previously.The lesser fluctuation at Well MAD3N9W-16.8a is explained by the presence of the adjacent drainageway and the well's proximity to Horseshoe Lake.

Ground-water levels in Well STC2N9W-26.8f2 (figure 12d) and Well MAD3N9W-8.5g1 (figure 12e) vary in an almost identical manner, probably because both wells are in urban areas. The presence of high-density buildings and large paved areas limits the area through which vertical recharge can occur. Also, as a result of the network of storm drainage in urban areas, potential recharge from precipitation is carried away quickly, resulting in moderate water-level changes. In contrast, water levels in Well STC2N10W-12.7g (figure 12f) are impacted heavily by pumpage and by river-stage levels. The resulting impact of these influences is an annual water-level change of 5 feet. During the period 1986-1990, pumpage increased approximately 13 percent over the previous five-year period at the National City pumping center and low river stages during the drought of 1988-1989, which contributed to water levels receding below the bottom of Well STC2N10W-12.7g from July 1988 to March 1990. Rapid and dramatic water-level changes occur at Well MAD3N10W-12.4f (figure 12g) and Well MAD5N9W-18.3c (figure 12h) because of the effect of fluctuations in the Mississippi River. Declining water levels during this same period reflect below-average precipitation and river stages during 1988 and 1989 in the hydrographs for Wells MAD3N10W-12.4f and MAD5N9W-18.3c. This downward trend in ground-water levels was reversed during 1990 when precipitation and Mississippi River stages were well above normal.

Figure 13(a-d) shows hydrographs of selected wells for the entire period of record. Well MAD3N8W-31.1a (figure 13a) reflects the slight downward trend of water levels in the Collinsville area as a result of the growing pumping cone. Wells MAD3N9W-16.8a (figure 13b), MAD3N10W-12.4f (figure 13c), and MAD5N9W-27.5a1 (figure 13d) indicate that the trend of continuously rising water levels, experienced in the area since 1965 because of the overall decrease in ground-water use and shifts in the distribution of pumpage, has ceased. From 1985-1990, hydrographs for these wells have shown a stabilized to a slight downward trend. Relatively consistent pumpage from 1981-1990 has led to these pumping centers having less influence on the surrounding water levels. The controlling factors in water-level trends between 1981 and 1990 appear to be precipitation and stream levels.

POTENTIOMETRIC SURFACE: NOVEMBER 1990

A potentiometric surface map (figure 14) was prepared from water levels measured in 269 wells during late October and early November 1990 when water levels are usually near minimum stages. Figure 15 provides locations of wells, and the appendix provides ground-water level data used to prepare the map. Tables 3 and 4 indicate surface water elevations used in preparing the potentiometric surface map.

Prior to development of large ground-water supplies, ground-water movement was toward the Mississippi River and other streams and lakes. During high river stages, flow was from the river. With the development of large ground-water supplies, however, the general pattern of ground-water flow has been toward the cones of depression created by pumpage or the Mississippi River and lakes and other streams. In places where cones of depression are near the river, hydraulic gradients from the river have been established and significant quantities of river water are diverted into the pumping centers.

The main features of the November 1990 potentiometric map (figure 14) are the deep cones of depression along the Mississippi River just south of Alton and near National City. The cone of depression at Alton was formed by pumping for dewatering during construction of the Mel Price Lock and Dam. The cone of depression near National City is the result of dewatering to maintain ground-water elevations below the highway surface in areas where the highway is depressed below the original land surface.

Other features include cones of depression associated mainly with industrial pumpage just south of the bluffs near Alton and at Wood River, Roxana, and Granite City. A cone of depression along the bluffs near Collinsville is the result of pumpage for municipal use. Withdrawals in the vicinity of Sauget were negligible in 1990. Consequently, the cone of depression associated with industrial pumpage at Sauget has disappeared, and ground-water movement in the vicinity was toward the river.

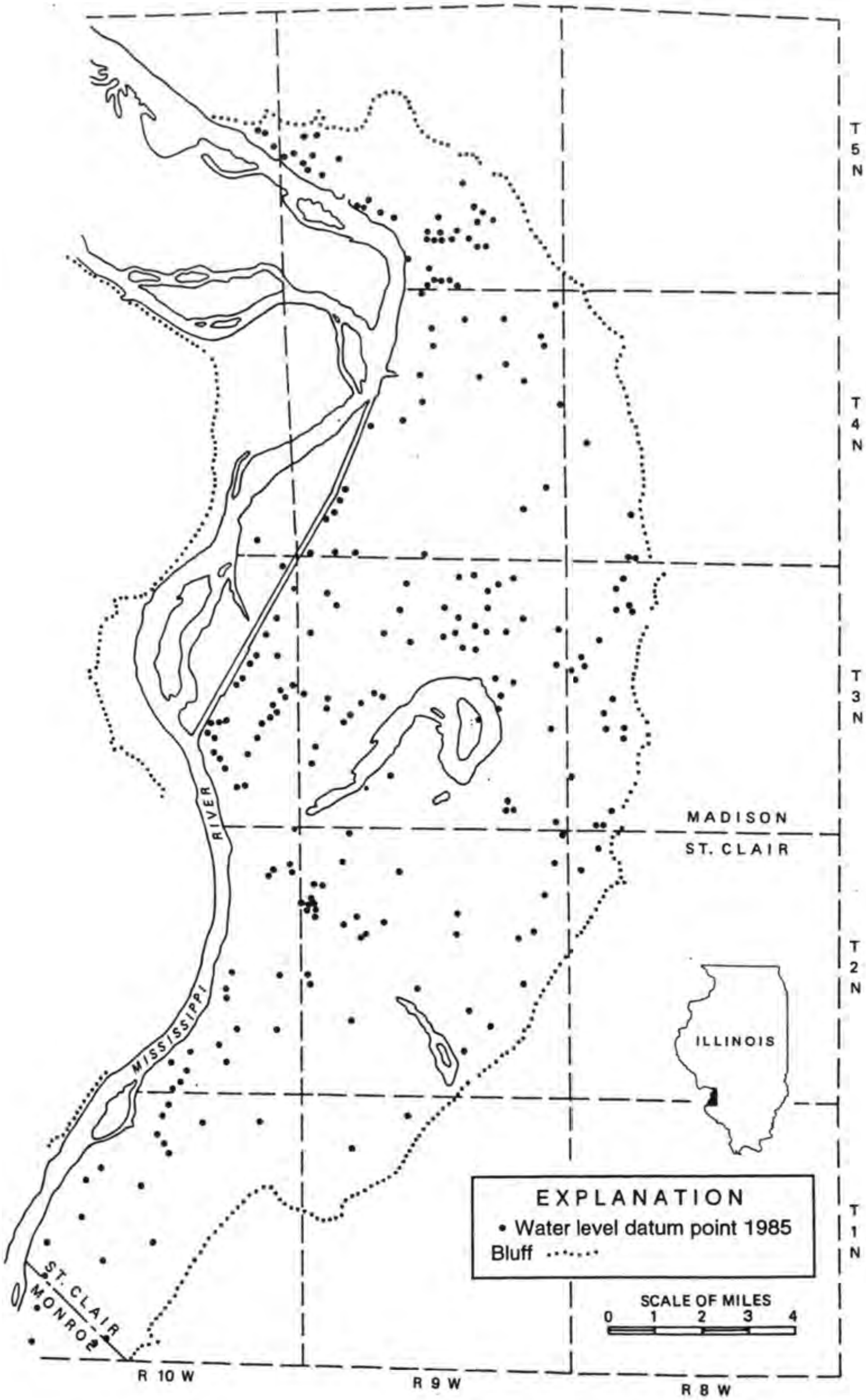


Figure 15. Location of datum points used for 1990 mass measurement

CHANGES IN GROUND-WATER LEVELS

November 1985-1990

Figure 16 shows ground-water level changes from November 1985-November 1990. Changes were estimated by comparing potentiometric surface maps for 1985 (figure 17) and 1990 (figure 14). Significant declines exceeding 25 feet occurred along the Mississippi River a few miles south of Alton adjacent to the Mel Price Lock and Dam as a result of dewatering during construction of the lock and dam. Ground-water level declines exceeded 5 feet in an area extending from Granite City to Sauget, and continuing in a narrow band south along the river to the edge of the study area. These changes were attributed to a significant change in river stage (figure 10) between November 1985 and November 1990. No changes were recorded in the vicinity of the main highway dewatering area near National City where pumpage is adjusted to maintain constant water levels. Ground-water levels were less than 5 feet below 1985 levels in the rest of the area except for a large area in the vicinity of Wood River and Roxana where declines exceeded 5 feet. These changes were attributed to below normal precipitation in 1988 and 1989 (figure 11). Although precipitation was above normal during 1990, ground-water levels had not recovered completely.

November 1966-1990

To show the impact of large declines in ground-water pumpage, a water-level change map for the period November 1966-1990 (figure 18) was estimated by comparing the potentiometric surface maps for 1966 (figure 19) and 1990 (figure 14). Ground-water pumpage was 108.1 mgd in 1966, near the peak of 111.0 mgd recorded in 1956 (Reitz, 1968). By 1990, ground-water pumpage declined to 58.7 mgd. Table 5 shows declines in pumping for each major pumping center. Pumping for dewatering during construction of the Mel Price Lock and Dam near Alton was not included in the Alton total because it is difficult to estimate and is only temporary.

Except for a narrow strip along the bluffs from Collinsville to just south of Cahokia Diversion Canal, an area in the vicinity of Alton, and a small area in the vicinity of East Carondelet along the Mississippi River, ground-water levels rose between November 1966 and November 1990, mainly because of the reduction in pumpage.

With the exception of the Alton area, ground-water levels in the vicinity of pumping centers rose during the 1966-1990 period. At Alton the impact of a large decline in estimated pumpage (7.6 mgd) was balanced by the dewatering pumpage at the Mel Price Lock and Dam and water levels not significantly different in 1990 than in previous years. The greatest recovery occurred at the Sauget pumping center where water levels rose more than 65 feet. Pumpage at Sauget for the period declined 27.3 mgd. Water-level recovery exceeded 10 feet at Wood River and exceeded 15 feet at Granite City and north of the National City pumping center. Because of the large quantities of ground water withdrawn for the highway dewatering system, ground-water level recovery was significantly less along interstate highways in the vicinity of National City as shown in figure 18. Recovery of water levels was less than 10 feet and in some areas less than 5 feet in a broad band along the interstate highway.

Ground-water levels declined in a narrow band along the bluffs from the Cahokia Diversion Channel to Collinsville. Declines also occurred along the Mississippi River south of Cahokia Canal and in small areas in Wood River and East Alton.

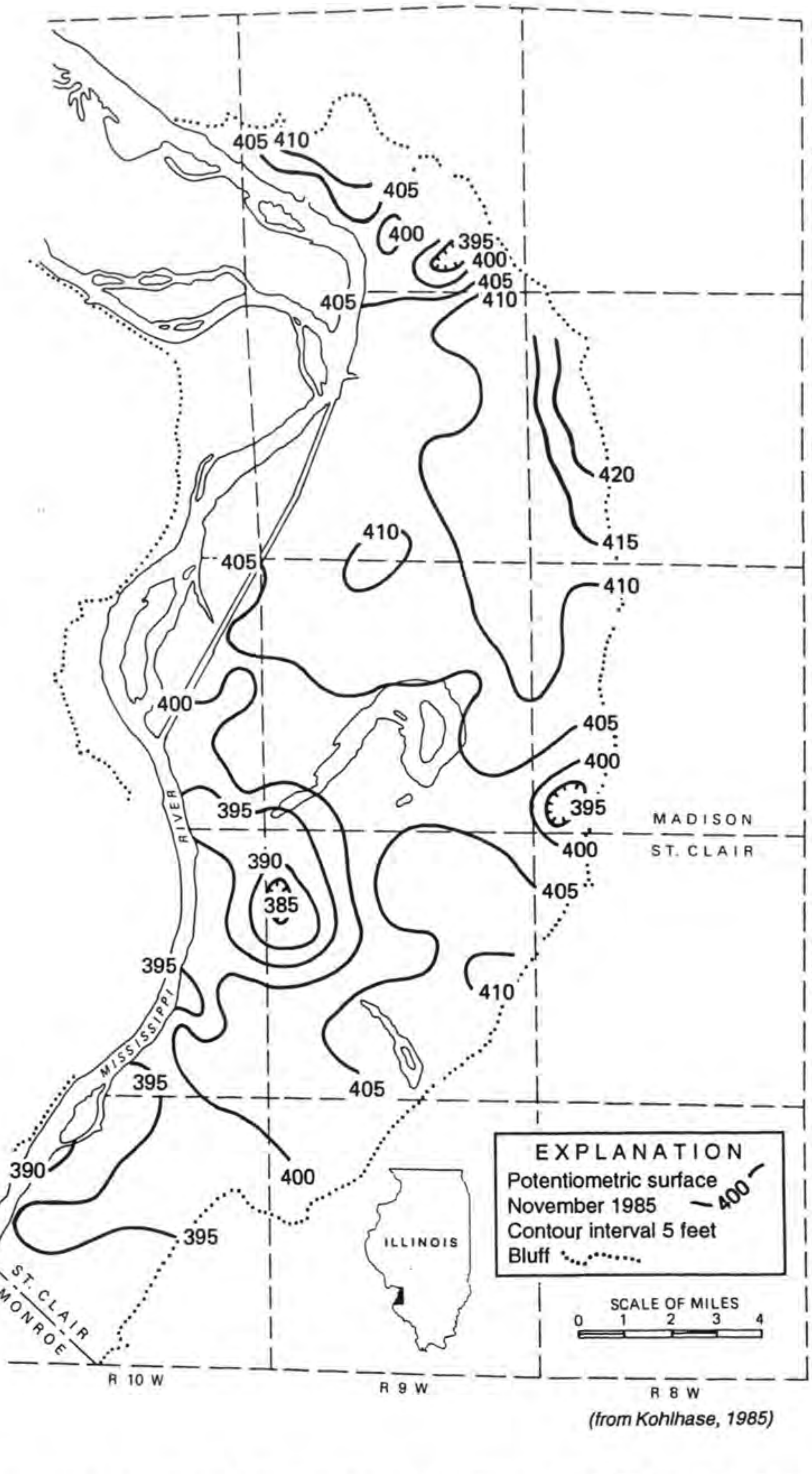


Figure 17. Appproximate elevation of potentiometric surface, November 1985 (from Kohlhase, 1985)

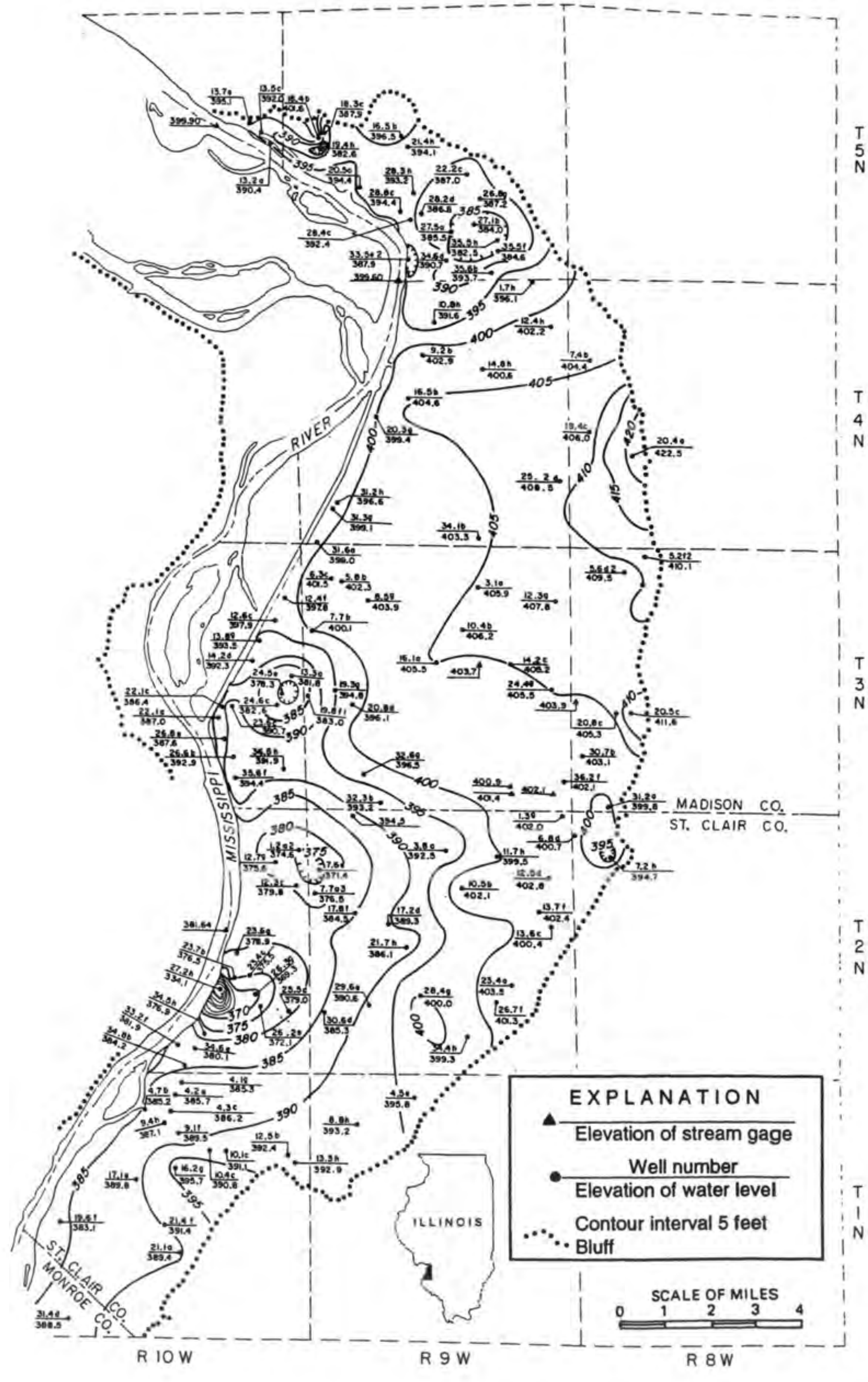


Figure 19. Approximate elevation of potentiometric surface for 1966 (from Reitz, 1968)

AREAS OF DIVERSION

Figure 20 shows boundaries of areas of diversion of pumping centers for November 1990. The boundaries delimit areas within which the general movement of ground water is toward pumping centers. In areas where ground-water levels are near the land surface, ground-water may discharge into streams, lakes, or both. It has been more difficult to determine areas of diversion of pumping centers because ground-water levels have recovered significantly in recent years. For this study only, areas of diversion that are easily recognizable on the potentiometric surface are shown.

Hydraulic gradients were established from the Mississippi River toward the pumping centers in the Alton and Wood River areas of diversion. As a result the river contributes a large part of the pumpage.

For the areas of diversion for Granite City, Venice, and National City, a ground-water divide exists between the pumping center and the river. It should be noted that the ground-water areas of diversion shown exist for only the period that water levels weremeasured. Areas of diversion may be distorted markedly by changes in river stage, particularly significant increases in stage and significant rainfall recharge events and significant changes and shifts in pumpage.

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APPENDIX B. WATER-LEVEL ELEVATIONS AND CHANGES IN THE METRO-EAST AREA, 1985-1990

County location	*Owner*	*Water-level elevation 1985 (ft)*	*Water-level elevation 1990 (ft)*	*Water-level change 1985-1990 (ft)*
Madison				
3N08W05.2d	SCHWARTZ		407.83	
3N08W05.2f2	V OF GLEN CARBON #2 (sealed > 1982)			
3N08W05.2f3	V OF GLEN CARBON #3 (sealed > 1982)			
3N08W05.4a1	V OF MARYVILLE #1		404.80	
3N08W05.4a2	V OF MARYVILLE #2		404.30	
3N08W05.4a3	V OF MARYVILLE #3		405.60	
3N08W05.4a4	MARYVILLE WELL FIELD - ME4		403.81	
3N08W05.4h	LOHR BROS CONST	413.23	403.52	-9.71
3N08W05.5e	V OF GLEN CARBON #6	409.99	409.12	-0.87
3N08W05.6d1	V OF GLEN CARBON #4	410.43	409.96	-0.47
3N08W05.6d2	V OF GLEN CARBON #5	411.06	410.08	-0.98
3N08W08.4g	KELLER #3			
3N08W08.6h	WILLAREDT, HARLEY	408.00	410.38	2.38
3N08W18.7e	ARLINGTON GOLF COURSE		408.95	
3N08W19.1f	FERD STRACKETJAHN	408.66	408.93	0.27
3N08W19.7e	HADLEY BRIDGE			
3N08W20.5a1	V OF TROY WELL #1	403.04	402.90	-0.14
3N08W20.5a2	V OF TROY WELL #2	401.94	402.98	1.04
3N08W20.5a3	V OF TROY WELL #3	403.88	402.31	-1.57
3N08W20.5a4	V OF TROY WELL #4		403.75	
3N08W20.5c	TED KOSTEN JR.		405.05	
3N08W20.8c	E FOURNIE	406.36	406.37	0.01
3N08W30.7b	V W ECKMANN	403.83	403.89	0.06
3N08W31.1a1	COLLINSVILLE OB WELL ID#1073	397.68	398.63	0.95
3N08W31.1a2	C OF COLLINSVILLE #7	390.25	390.05	-0.20
3N08W31.1a3	C OF COLLINSVILLE #8		388.00	
3N08W31.1a4	C OF COLLINSVILLE #11		392.43	
3N08W31.2a1	C OF COLLINSVILLE #9		391.66	
3N08W31.2a2	C OF COLLINSVILLE #10	390.44		
3N08W32.8d	WATSON		399.72	
3N09W03.1a	CARL ELLIS	408.29	406.95	-1.34
3N09W04.5e1	C OF GRANITE CITY P-2			
3N09W04.5e2	MARYVILLE SCHOOL - ME1		408.40	
3N09W06.1a	HERBERT BISCHOFF #1	408.56	406.27	-2.29
3N09W06.3c	HERBERT BISCHOFF #2	407.81	405.46	-2.35
3N09W07.6d	A O SMITH CO WELL A	407.07	403.73	-3.34
3N09W08.1d	C OF GRANITE CITY P-5	408.52	406.91	-1.61
3N09W09.4c1	MIKE GRAVES			
3N09W09.4c2	PARKVIEW SCHOOL - ME3		408.53	
3N09W10.2a	WILBERT ENGELKE (S of tracks)	406.58	406.15	-0.43
3N09W10.4b	WILBERT ENGELKE (destroyed > 1985)	408.03		
3N09W10.4g1	C OF GRANITE CITY P-4			
3N09W10.4g2	C OF GRANITE CITY P-4A	408.72	407.96	-0.76
3N09W10.5d	GOLF COURSE (THE REGENCY)		418.28	
3N09W10.6c	M ORASCO			
3N09W12.3g	CHARLES LUEHMANN	413.42	412.84	-0.58

APPENDIX B. (Continued)

County location	Owner	Water-level elevation 1985 (ft)	Water-level elevation 1990 (ft)	Water-level change 1985-1990 (ft)
Madison				
3N10W23.7c	E ST L D&L DIS RW20	398.05	393.21	-4.84
3N10W24.1c	GRANITE CY STEEL #2		391.96	
3N10W24.3h	PRAIRE FARMS DAIRY	401.27	398.00	-3.27
3N10W24.5e	GRANITE CY STEEL #14	396.47	393.52	-2.95
3N10W24.5f	GRANITE CY STEEL #16	397.01	394.22	-2.79
3N10W24.6d	GRANITE CY STEEL #15	401.41	398.50	-2.91
3N10W24.7c	GRANITE CY STEEL #17	399.33	396.42	-2.91
3N10W25.8h	COVALCO	401.10	398.40	-2.70
3N10W26.2e1	DUNBAR SCHOOL - ME16		398.76	
3N10W26.6b	E ST L D&L DIS RW78	397.44	392.73	-4.71
3N10W26.7d	E ST L D&L DIS RW70		392.84	
3N10W26.8e	E ST L D&L DIS RW64	398.85	394.16	-4.69
3N10W26.8h	E ST L D&L DIS RW53	397.72	393.09	-4.63
3N10W35.3f	IDOT DEWATERING #4	394.46		
3N10W35.4f	IDOT DEWATERING #1	394.03		
3N10W35.6f	E ST L D&L DIS RW96			
3N10W35.6g	E ST L D&L DIS RW91	397.28		
3N10W35.6h	E ST L D&L DIS RW87	396.87		
3N10W36.5g	MAD INDUS COMPLEX#11	401.10		
3N10W36.5h	LACLEDE STEEL CO #9	400.36		
4N08W17.8b1	SIU EDWRD WELL 1	425.57		
4N08W17.8b2	SIU EDWRD WELL 2	421.13	415.03	-6.10
4N08W18.4c	BROCKMEIR WELL 2	416.36		
4N08W19.4e	I.J. HITTNER		409.85	
4N08W20.4a	BROCKMEIR WELL 1	424.78		
4N08W20.5d	SIU WELL 3	418.91		
4N08W29.4a	OTTO BAUMANN	416.32	414.06	-2.26
4N08W32.3a	VERNON KELLER WELL 1	421.09	413.55	-7.54
4N08W32.4a	VERNON KELLER WELL 2		412.31	
4N09W01.2e	LOSCH FARMS	417.87	410.22	-7.65
4N09W01.7h1	MARRIN DENTON			
4N09W02.3b	VIL OF ROXANA	413.09	407.18	-5.91
4N09W03.2b	EXPLORER PIPELINE CO	409.88	405.49	-4.39
4N09W03.2g	SHELL OIL CO	402.11		
4N09W03.6f	SHELL OIL CO			
4N09W04.2g3	VIL OF HARTFORD WELL 1		397.99	
4N09W04.2g4	VIL OF HARTFORD WELL 2	403.04	396.34	-6.70
4N09W04.2g5	VIL OF HARTFORD WELL 4		404.45	
4N09W04.3f	CITY OF HARTFD WELL 3	404.58	399.56	-5.02
4N09W04.5f	NAT MARINE SERVICE WELL 1			
4N09W04.6e	NAT MARINE SERVICE WELL 2			
4N09W04.7h	HARTFORD, IL RM196.8			
4N09W09.2b	HOEHN WELL (destroyed>1980)			
4N09W10.8e	CONOCO PIPELINE CO		405.65	
4N09W10.8h	HARTFORD TERMINAL	408.01	404.53	-3.48
4N09W11.3b1	ROXANA DISTR SYSTEM #8			
4N09W11.3b2	ROXANA DISTR SYSTEM #9	410.36		
4N09W11.3b3	ROXANA DISTR SYSTEM #10		405.69	

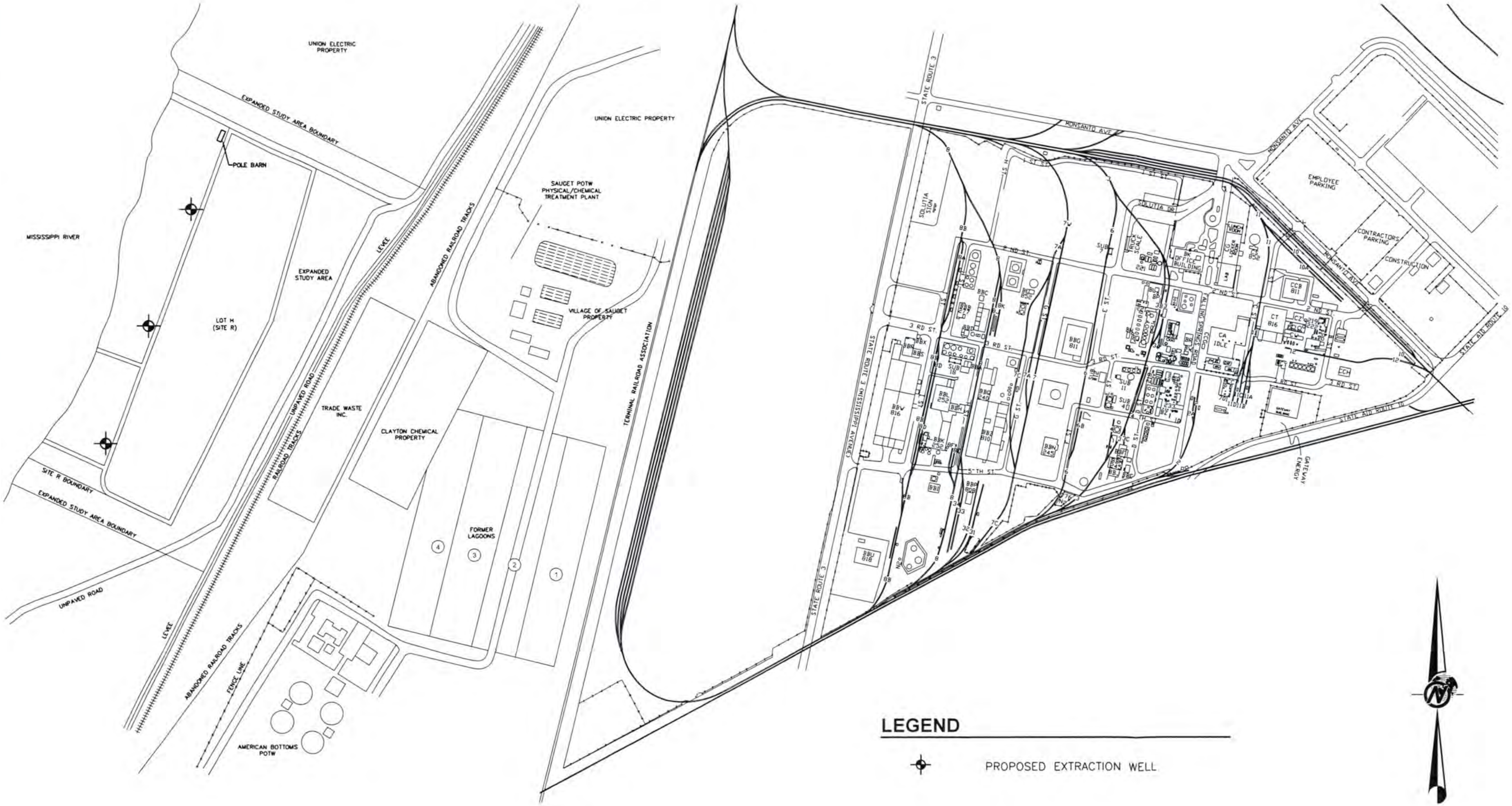
APPENDIX B. (Continued)

County location	Owner	Water-level elevation 1985 (ft)	Water-level elevation 1990 (ft)	Water-level change 1985-1990 (ft)
Madison				
5N09W20.4h3	CY OF E ALTON #3	404.41	392.50	-11.91
5N09W20.4h4	CY OF E ALTON #4		394.21	
5N09W20.4h5	CY OF E ALTON #5		398.00	
5N09W20.5a	WOOD RIVER D&L DIS RW105			
5N09W20.8g1	AIRCO INDUST GAS #1			
5N09W20.8g2	AIRCO INDUST GAS #2		397.80	
5N09W21.5c	DOME RAILWAY SERV #1	411.30		
5N09W21.5h1	CY OF E ALTON #15			
5N09W21.5h2	CY OF E ALTON #16			
5N09W21.5h3	CY OF E ALTON #19			
5N09W21.5h4	CY OF E ALTON #11			
5N09W22.2c1	VIL OF BETHAL #1			
5N09W22.2c2	VIL OF BETHAL #2			
5N09W22.2c3	VIL OF BETHAL #3			
5N09W22.2c6	VIL OF BETHAL #6	403.34	392.45	-10.89
5N09W22.2c7	VIL OF BETHAL #7	401.78	391.17	-10.61
5N09W22.2c8	VIL OF BETHAL #8	401.78	392.32	-9.46
5N09W22.2c9	VIL OF BETHAL #9	402.12	391.48	-10.64
5N09W22.2c10	VIL OF BETHAL #10	399.20	388.50	-10.70
5N09W22.2c11	VIL OF BETHAL #11		392.03	
5N09W22.2c12	VIL OF BETHAL #12	399.42	389.92	-9.50
5N09W22.4e	CY OF WOOD RIVER, BELK PARK			
5N09W26.7f	CY OF WOOD RIVER #17	405.89	397.60	-8.29
5N09W26.8d1	VIL OF ROXANA #6			
5N09W26.8d2	WOOD RIVER D&L DIS #136	404.38	398.68	-5.70
5N09W26.8e	VIL OF ROXANA #7			
5N09W26.8g1	CY OF WOOD RIVER #12	408.18		
5N09W26.8g2	CY OF WOOD RIVER #15	405.94		
5N09W26.8g3	CY OF WOOD RIVER #18		397.06	
5N09W27.1b2	VIL OF ROXANA #3			
5N09W27.1b4	VIL OF ROXANA #5			
5N09W27.5a1	MARATHON PLINE S WELL	400.85	395.04	-5.81
5N09W27.5a2	MARATHON OIL N WELL	400.68	393.02	-7.66
5N09W27.7a	AM OIL CO WR REF #60	395.35	390.71	-4.64
5N09W27.7b	AM OIL CO WR REF #42		389.94	
5N09W27.7e1	AM OIL CO WR REF #50	402.68	394.87	-7.81
5N09W27.7e2	AM OIL CO WR REF #51	395.64		
5N09W27.7e3	AM OIL CO WR REF #53	402.91	397.06	-5.85
5N09W27.8a1	AM OIL CO WR REF #58	396.38	388.86	-7.52
5N09W27.8a2	AM OIL CO WR REF #61	396.53	392.16	-4.37
5N09W27.8b1	AM OIL CO WR REF #56	397.38		
5N09W27.8b2	AM OIL CO WR REF #55	411.62		
5N09W27.8b3	AM OIL CO WR REF #65			
5N09W27.8c	AM OIL CO WR REF #33	398.64		
5N09W27.8d1	AM OIL CO WR REF #30			
5N09W27.8d2	AM OIL CO WR REF #52			
5N09W28.1a1	AM OIL CO WR REF #59	398.96	393.09	-5.87
5N09W28.1a2	AM OIL CO WR REF #62	398.37		

APPENDIX B. (Continued)

County location	*Owner*	*Water-level elevation 1985 (ft)*	*Water-level elevation 1990 (ft)*	*Water-level change 1985-1990 (ft)*
Madison				
5N10W13.1a1	LACLEDE STL CO (ALTON) #1		394.80	
5N10W13.1a2	LACLEDE STL CO (ALTON) #3	407.10		
5N10W13.1b	LACLEDE STL CO (ALTON) #2	401.25		
5N10W13.2a1	WOOD RIVER D&R DIS RW41X		403.92	
5N10W13.2a2	WOOD RIVER D&R DIS RW42X	407.09	403.44	-3.65
5N10W13.4c1	OWENS IL GLASS CO #1			
5N10W13.4c3	OWENS IL GLASS CO #3	402.03		
5N10W13.4c6	OWENS IL GLASS CO #6	402.03		
5N10W13.4c7	OWENS IL GLASS CO #7	401.18		
5N10W13.4c8	OWENS IL GLASS CO - COE WELL		404.80	
5N10W13.5c	WOOD RIV D&L DIS RW20	408.84	411.17	2.33
5N10W13.5d1	WOOD RIV D&L DIS RW16			
5N10W13.5d2	WOOD RIV D&L DIS RW18	410.50		
5N10W14.4e	LOCK & DAM #26			
5N10W24.1h	WOOD RIV D&L DIS RW51	407.50	397.69	-9.81
St. Clair				
1N09W04.5e	E WESTERHEIDE			
1N09W04.6f1	LaLUMIER SCHOOL - ME22		402.50	
1N09W06.1e	SWS PIEZOMETER			
1N09W08.8h	VA RISTER	401.28	400.11	-1.17
1N10W01.8d1	CAHOKIA HIGH SCHOOL - ME13		397.33	
1N10W02.8e	SWS PIEZOMETER			
1N10W03.3c1	HUFFMAN SCHOOL - ME14		395.12	
1N10W04.1g	E ST L D&L DIS RW196	393.90	391.05	-2.85
1N10W04.2e	E ST L D&L DIS RW207	393.36	389.46	-3.90
1N10W04.3b	E ST L D&L DIS RW237		389.86	
1N10W04.3c	E ST L D&L DIS RW223			
1N10W04.7b	PRAIR DUP D&L RW23	390.81	386.95	-3.86
1N10W08.2h	PRAIR DUP D&L RW28	390.33	388.41	-1.92
1N10W08.5c	PRAIR DUP D&L RW34	390.84	389.45	-1.39
1N10W08.7a	PRAIR DUP D&L RW45	390.36	389.02	-1.34
1N10W09.1f	E ST L D&L DIS RW262	394.21	391.99	-2.22
1N10W09.2h	E ST L D&L DIS RW251	395.36	392.09	-3.27
1N10W09.4h	PRAIR DUP D&L RW15	392.92	389.01	-3.91
1N10W10.1c	E ST L D&L DIS RW273			
1N10W10.4c	E ST L D&L DIS RW263			
1N10W12.5b	E ST L D&L DIS RW278	398.61	398.16	-0.45
1N10W13.3h	E ST L D&L DIS RW286	397.74	398.21	0.47
1N10W16.2g	WALTER DRESCHER	397.63		
1N10W16.6h	OSCAR KELLING	395.60	394.74	-0.86
1N10W17.1e	OSCAR KELLING			
1N10W17.5g	D CHARTRAND			
1N10W17.8b	D CHARTRAND	399.64	397.23	-2.41
1N10W19.6f	PRAIR DUP D&L RW46	390.40	386.77	-3.63
1N10W20.4c	C LINDHORST	390.91	389.22	-1.69
1N10W20.5f	D CHARTRAND			
1N10W20.6a	D CHARTRAND			

County location	Owner	Water-level elevation 1985 (ft)	Water-level elevation 1990 (ft)	Water-level change 1985-1990 (ft)
St. Clair				
2N09W12.5d1	BILL HENSON (ex VERNON STAFFORD)	407.57	406.34	-1.23
2N09W12.5d2	BILL HENSON #2		407.75	
2N09W13.7f	J COURTNEY	408.14	406.69	-1.45
2N09W14.2e	BLUFFVIEW PARK - ME21		408.68	
2N09W14.3d	NAGLE			
2N09W14.3f	C WEISSERT #3	405.47		
2N09W14.6h	FRANK TOJO			
2N09W15.5e1	C WEISSERT #1	402.10	401.39	-0.71
2N09W15.5e2	A WEISSERT #1	408.63	406.69	-1.94
2N09W16.7a	ESL CASTINGS CO			
2N09W17.2g	CY OF E ST L JONES P	397.63	397.02	-0.61
2N09W17.7h1	CHAS PFIZER INC #12	386.69	384.95	-1.74
2N09W17.7h2	CHAS PFIZER INC #14	389.39	389.67	0.28
2N09W18.1g	ATHLETIC FIELD - ME9		391.53	
2N09W18.6h1	IDOT DEWAT I64 #5	386.37	382.09	-4.28
2N09W18.6h2	IDOT DEWAT I64 #13	385.26	379.70	-5.56
2N09W18.6h3	IDOT DEWAT I64 #14	386.96		
2N09W18.6h4	IDOT DEWAT I64 #15	387.09	383.77	-3.32
2N09W19.7d1	OBER NESTOR GLASS CO (SE WELL)	400.53	399.23	-1.30
2N09W19.7d2	OBER NESTOR GLASS CO (NW WELL)	399.65	398.24	-1.41
2N09W19.8f1	CERTAIN-TEED PROD #1		394.40	
2N09W19.8f2	CERTAIN-TEED PROD #2	396.28	394.85	-1.43
2N09W21.4d	ESL HIGH SCHOOL - ME20		404.37	
2N09W23.1e	RICHARD POPP	410.88	409.98	-0.90
2N09W24.6e	MITCHELLS	410.42		
2N09W26.7e	SWS #2	408.70	407.81	-0.89
2N09W27.3g2	KENNEDY-KING SCHOOL - ME11		406.93	
2N09W27.8g	HOLTEN ST PK (GRAND MARIOS)			
2N09W28.3a	De MANGE			
2N09W28.4g	HOLTEN ST PK (GRAND MARIOS)	408.91	408.32	-0.59
2N09W29.8f1	CHEMTEK PRODS INC #14			
2N09W29.8f2	CHEMTEK PRODS INC #3	405.01	403.97	-1.04
2N09W29.8f3	CHEMTEK PRODS INC #7	405.57		
2N09W29.8f4	CHEMTEK PRODS INC #10			
2N09W29.8f5	CHEMTEK PRODS INC #12			
2N09W29.8f6	CHEMTEK PRODS INC #16		403.66	
2N09W33.1e	VINCE DEMANGE			
2N09W34.4h	H W THOMAS	407.57	406.07	-1.50
2N10W01.2h	USS AG CHEMICALS	390.44	388.32	-2.12
2N10W01.3a	ARMOUR AND CO WELL #2	392.19	390.73	-1.46
2N10W11.4e1	E ST L D&L DIS RW105			
2N10W11.4e2	E ST L D&L DIS RW108			
2N10W12.2h3	NATIONAL CY COLD STRG #6			
2N10W12.3g	SWIFT AND CO #17			
2N10W12.3h1	ARMOR AND CO WELL #4	392.87	392.85	-0.02
2N10W12.3h2	SWIFT AND CO #18	390.88		
2N10W12.6h1	ROYAL PACKING CO #1	395.69		
2N10W12.6h2	ROYAL PACKING CO #2	396.02	389.58	-6.44



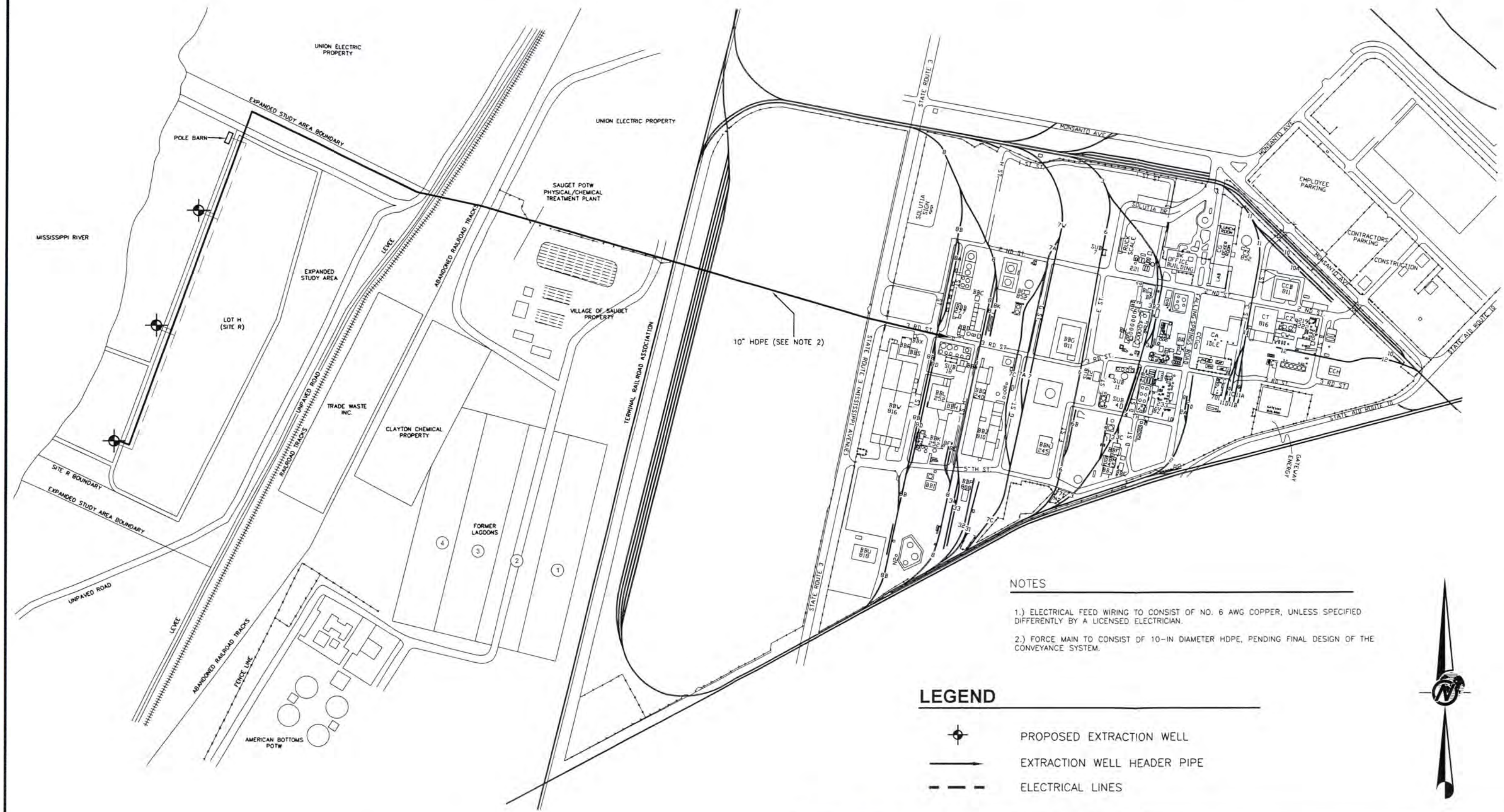
LEGEND

PROPOSED EXTRACTION WELL

0 500 1000

APPROXIMATE SCALE IN FEET

Golder Associates, St. Louis, Missouri	SCALE AS SHOWN	TITLE	
	DATE OCTOBER 2001	**SITE MAP**	
	DESIGN DESIGN		
	CADD MSL	PROJECT SOLUTIA GROUNDWATER SYSTEM DESIGN	DWG/FIG No.
FILE No. 9665.001	CHECK JRS		
PROJECT No. 013–9665 REV. 2	REVIEW JRS	SAUGET, ILLINOIS	1



NOTES

1.) ELECTRICAL FEED WIRING TO CONSIST OF NO. 6 AWG COPPER, UNLESS SPECIFIED DIFFERENTLY BY A LICENSED ELECTRICIAN.

2.) FORCE MAIN TO CONSIST OF 10-IN DIAMETER HDPE, PENDING FINAL DESIGN OF THE CONVEYANCE SYSTEM.

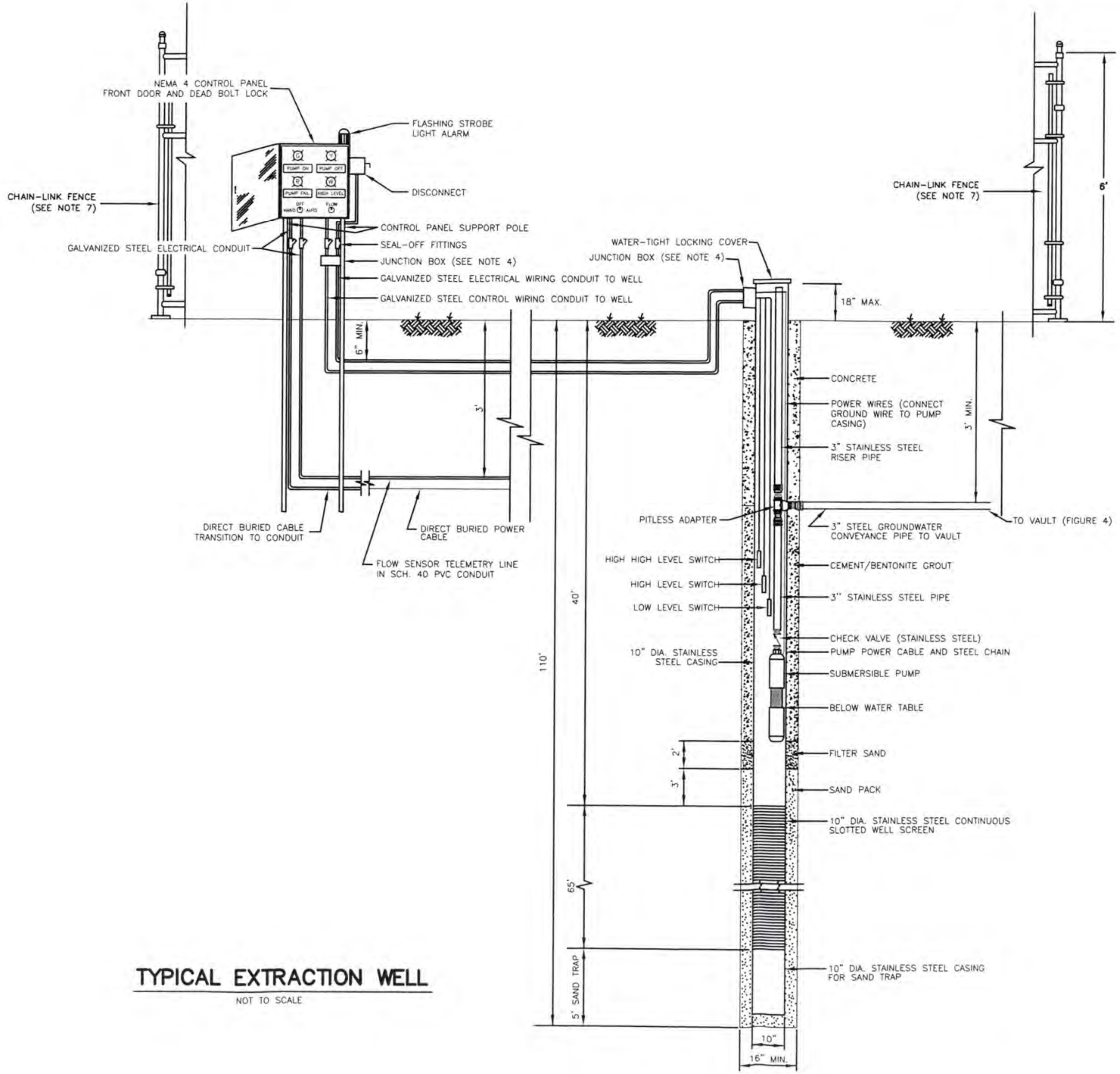
LEGEND

- PROPOSED EXTRACTION WELL
- EXTRACTION WELL HEADER PIPE
- ELECTRICAL LINES

0 500 1000

APPROXIMATE SCALE IN FEET

Golder Associates, St. Louis, Missouri	SCALE AS SHOWN	TITLE	PIPING AND ELECTRICAL RUN
	DATE OCTOBER 2001		
	DESIGN DESIGN	PROJECT	SOLUTIA GROUNDWATER SYSTEM DESIGN
	CADD MSL		
FILE No. 9665.002	CHECK JRS		SAUGET, ILLINOIS
PROJECT No. 013-9665 REV. 3	REVIEW JRS	DWG/FIG No.	2

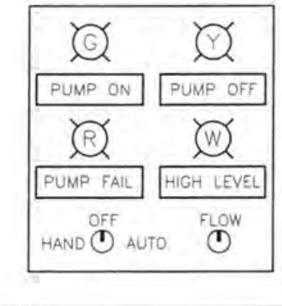


TYPICAL EXTRACTION WELL

NOT TO SCALE

DETAIL OF CONTROL PANEL

NOT TO SCALE



NOTES

1.) TYPICAL WELL CONSTRUCTION. ACTUAL PLACEMENT OF WELL SCREEN, CASING AND SAND PACK TO BE MODIFIED BY REMEDIAL DESIGNER BASED ON FIELD CONDITIONS.

2.) COMPLETION OF EXTRACTION WELLS SHALL BE UNDER THE DIRECT SUPERVISION OF THE REMEDIAL DESIGNER OR OTHER QUALIFIED PROFESSIONAL DESIGNATED BY SOLUTIA. THE FINAL COMPLETION OF EXTRACTION WELLS WILL BE BASED ON ACTUAL CONDITIONS ENCOUNTERED DURING WELL CONSTRUCTION.

3.) SUPPORT PUMP AND DISCHARGE PIPING FROM TOP. DO NOT ALLOW PITLESS ADAPTER TO SUPPORT PUMP UNLESS PITLESS ADAPTER IS CONCRETED IN PLACE. PROVIDE STAINLESS STEEL CHAIN TO ATTACH TO PUMP.

4.) ALL WIRING BETWEEN THE EXTRACTION WELLS AND CONTROL PANEL SHALL BE SUBMERSIBLE. CONNECT WIRES TO WELL CONDUIT USING SEAL TIGHT CONNECTORS. CONNECTOR SHALL HOLD FLOATS IN PLACE. PROVIDE ELECTRICAL BOX AT GROUND SURFACE SIZED TO STORE A MINIMUM OF 5 FEET OF EXCESS WIRES (SLACK) COILED NEATLY IN JUNCTION BOX FOR ADJUSTING DEPTH IN WELLS.

5.) ALL ELEVATIONS ARE APPROXIMATE. "HIGH HIGH LEVEL" FLOAT PRELIMINARILY LOCATED AT SAME ELEVATION AS NORMAL GROUNDWATER ELEVATION (NO PUMPING). HIGH LEVEL (PUMP ON) FLOAT PRELIMINARILY LOCATED 2 FT. BELOW "HIGH HIGH LEVEL" FLOAT. LOW LEVEL (PUMP OFF) FLOAT PRELIMINARILY SET 77 FT. ABOVE BOTTOM OF EXTRACTION WELL, ASSUMING PUMP IS PLACED 75 FT. FROM BOTTOM OF EXTRACTION WELL. ELEVATION OF ALL FLOAT SWITCHES MAY BE FIELD ADJUSTED DURING OPERATION FOR OPTIMIZATION OF SYSTEM. ALL PUMPS HAVE 10 HORSEPOWER MOTORS.

6.) AIR TUBE TO BE INSTALLED IN EACH WELL IN ORDER TO GAUGE DRAWDOWN, 6 INCHES FROM TOP OF PLATE.

7.) CONTRACTOR SHALL INSTALL A CHAIN LINK FENCE ENCLOSURE AROUND THE PERIMETER OF EACH WELL AS SPECIFIED IN FIGURE 5.

8.) SWITCH CABLE ADJUSTMENTS LOCATED 6 INCHES BELOW TOP OF LOCKING COVER.

REV	DATE	DES	REVISION DESCRIPTION	CADD	CHK	RVW
3	01/24/02	JRS	TEXT EDITS	JAP	JRS	FMB
2	11/27/01	JRS	FENCE ADDED	JAC	JRS	FMB
1	10/22/01	JRS	TEXT EDITS	JAC	JRS	FMB

PROJECT

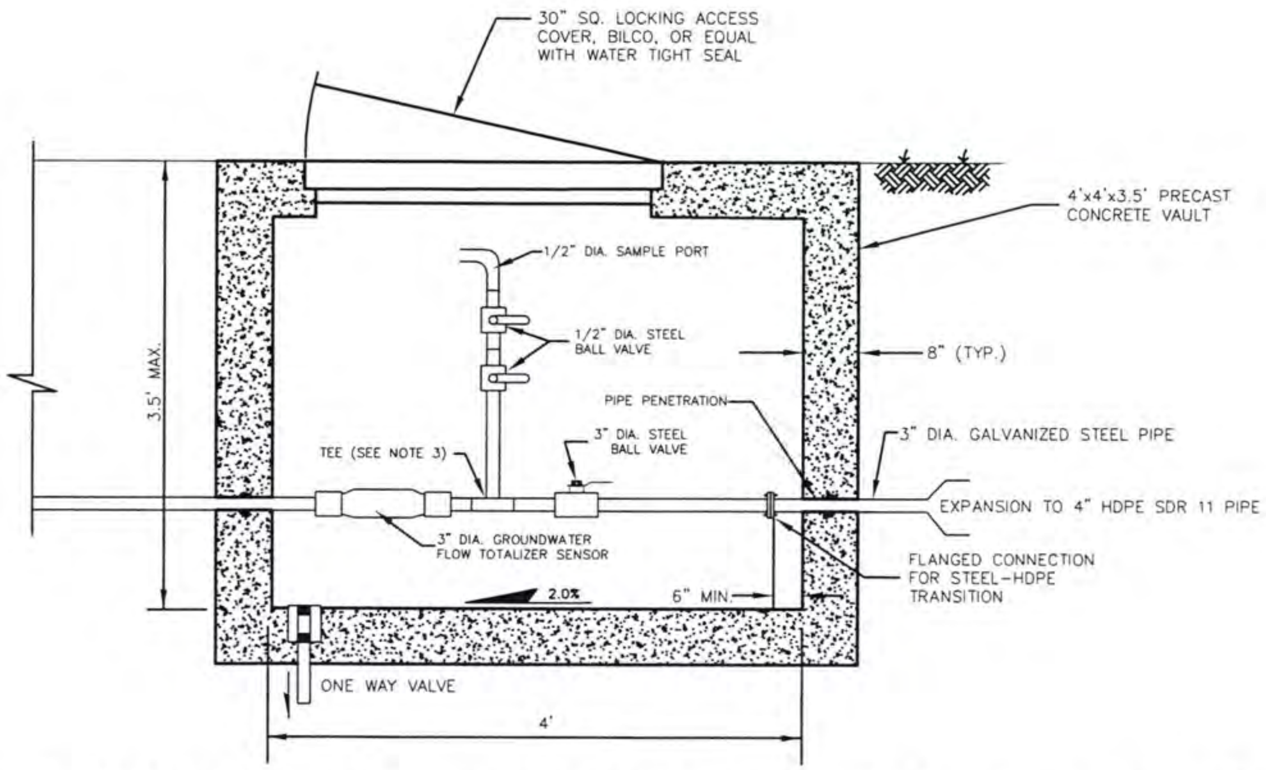
SOLUTIA EXTRACTION WELL SYSTEM

TITLE

GROUNDWATER EXTRACTION SYSTEM DETAILS

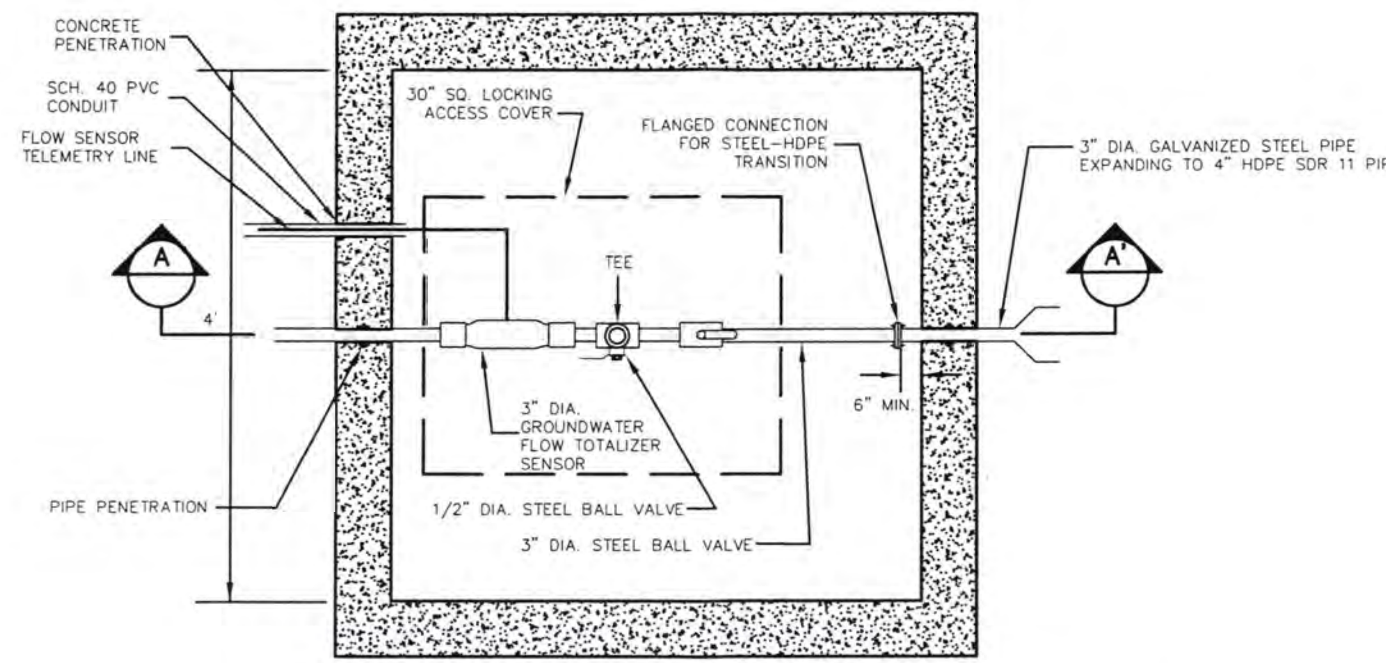
Golder Associates
St. Louis, Missouri

PROJECT No.	013-9665	FILE No.	9665.003
DESIGN	DNB	09/04/98	SCALE: AS SHOWN REV. 3
CADD	MRM	09/26/01	DWG/FIG No.
CHECK	JRS	10/22/01	
REVIEW	FMB	10/22/01	3



FLOW TOTALIZER AND SAMPLING VAULT – PROFILE VIEW A–A'

NOT TO SCALE



FLOW TOTALIZER AND SAMPLING VAULT – PLAN VIEW

NOT TO SCALE

NOTES

1.) ALL WIRING BETWEEN THE FLOW SENSOR AND CONTROL PANEL SHALL BE SUBMERSIBLE.

2.) FOR PRE-CAST CONCRETE, CONTRACTOR SHALL FOLLOW THE INSTRUCTIONS OF THE MANUFACTURER WHICH HAVE ALREADY BEEN FAVORABLY REVIEWED BY THE REMEDIAL DESIGNER.

3.) TEE JOINT TO BE AT LEAST FIVE INCHES FROM GROUNDWATER FLOW TOTALIZER SENSOR.

REV	DATE	DES	REVISION DESCRIPTION	CADD	CHK	RVW
3	01/24/02	JRS	EDITS	JAP	JRS	FMB
2	11/27/01	JRS	SAMPLE PORT MOVED	JAC	JRS	FMB
1	10/22/01	JRS	EDITS	JAC	JRS	FMB

PROJECT

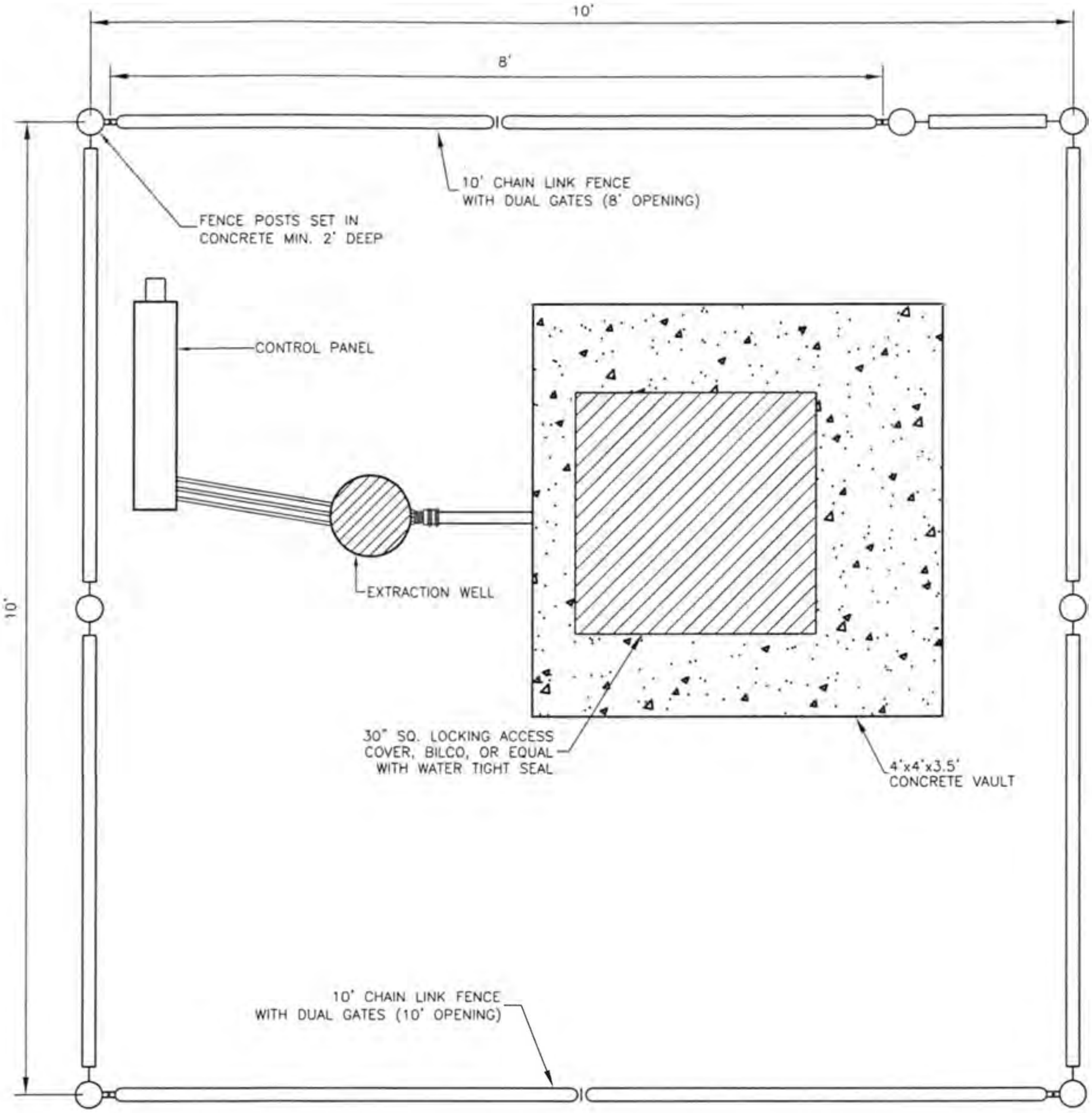
SOLUTIA EXTRACTION WELL SYSTEM

TITLE

VAULT DETAILS

Golder Associates
St. Louis, Missouri

PROJECT No.		013-9665	FILE No.	9665.004
DESIGN	DNB	09/04/98	SCALE: AS SHOWN	REV. 3
CADD	MRM	09/26/01	DWG/FIG No.	4
CHECK	JRS	10/22/01		
REVIEW	FMB	10/22/01		



NOTE

1.) CONTRACTOR SHALL INSTALL A CHAIN LINK FENCE ENCLOSURE AROUND THE PERIMETER OF EACH WELL. CONTRACTOR SHALL SUPPLY DUAL GATES WITH PADLOCKING LATCH FOR EACH ENCLOSURE AND POSITIONED TO ALLOW DIRECT ACCESS TO THE VAULT AND INTERNAL COMPONENTS. CHAIN LINK FENCE NOT SHOWN ON ALL DETAILS FOR CLARITY.

*NOT TO SCALE

Golder Associates
St. Louis, Missouri

SCALE	AS SHOWN	TITLE	TYPICAL WELL ENCLOSURE PLAN VIEW
DATE	NOV. 01		
DESIGN	FMB		
CADD	JAC	PROJECT	SOLUTIA EXTRACTION WELL SYSTEM
CHECK	JRS		Sauget, Illinois
REVIEW	FMB	DWG/FIG No.	5

FILE No.

PROJECT No. 013–9665 REV. 1

SOLUTIA - 069

"Hiller, Robert J"
<rjhill1@solutia.com>
02/08/02 09:12 AM

To: Kenneth Bardo cc: "Faust, Alan G"
Subject: Revised MCB Recovery RAP

Ken,

The attached Word file contains the revised remedial action plan for the continuation of the MCB recovery
project. We will be dismantling the current system and installing the new equipment beginning next week.
We have studied this strategy over the past few months and I feel that this is the best approach considering that:

1) only a single well volume of MCB can be removed in a pumping period.

2) the period of time needed for the wells to recover a volume of MCB.

The removal and recovery rates are impacted by the low flow rate of the MCB.

By employing a routine low flow pumping strategy we can maximize the amount of MCB removed from the
area.

Please contact me if you have any questions or comments regarding this action plan.

I will be forwarding a hard copy for your files shortly.

I hope to see you next week at the RCRA conference.

Thanks

Bob Hiller

<<Solutia (MCB Recovery) RAP.doc>> <<MCB Recovery Pump System.pdf>>

Solutia (MCB Recovery) RA MCB Recovery Pump Syste

February 6, 2002

Mr. Bob Hiller
Solutia, W.G. Krummrich Plant
500 Monsanto Avenue
Sauget, IL 62206-1198

Subject: **MCB Recovery Remedial Action Plan (RAP)**
Solutia, W.G. Krummrich Plant
Sauget, IL
<u>URS Project No. 23-20010023.01</u>

Dear Mr. Hiller:

URS Corporation (URS) is submitting this Remedial Action Plan (RAP) to identify the activities required to implement and operate a low-flow liquid pumping system to recover free product Monochlorobenzene (MCB) at the W.G. Krummrich (WGK) plant located in Sauget, IL. This RAP provides background information and the scope of work for the project.

BACKGROUND

Solutia reported a release of MCB to the Illinois Environmental Protection Agency (IEPA) on January 8, 2001. The release resulted in the migration of MCB to the subsurface in a process area within the WGK plant. Solutia requested URS to initiate investigation of the MCB release on January 15, 2001. The results of this investigation were reported to the IEPA at a meeting on January 25, 2001. At the meeting, Solutia proposed a pilot test/recovery approach and followed up in writing on February 7, 2001. IEPA approved the construction and operation of the pilot test/recovery system on March 14, 2001.

The system was constructed and pilot testing was performed from March 29 through April 3, 2001. Based on the pilot test results, URS concluded that a vacuum could not be effectively applied to the formation due to the presence of loose material and voids in the subsurface. Therefore, a Dual Phase Vapor Extraction (DPVE) system was deemed too inefficient to be implemented on a full-scale basis.

However, the pilot test results indicated that a continuous low-flow liquid pumping system could collect free product more efficiently. In addition, the system would eliminate unnecessary energy consumption and generation of large volumes of vapor phase Granulated Activated Carbon (GAC) for treatment and/or disposal. The system was constructed from May 29 through May 31, 2001 and consisted of a nitrogen driven pneumatic diaphragm pump connected to recovery well RW-1 by teflon hosing and discharging into the 20,000 gallon steel storage tank previously used for the DPVE pilot test. This system was placed in operation on May 31, 2001 and was operated through July 18, 2001. It was noted that the pump had a tendency to shut off by itself at low operating flowrates and that the recharge rate of recoverable free product at RW-1 had decreased.

URS recommended switching from continuous low-flow liquid pumping to periodic low-flow liquid pumping at RW-1. As well, URS recommended expansion of the pumping system to existing piezometers and proposed additional recovery wells (RW-2 & 3) (**Figure 1**). Piezometers PZ-1, 4, & 9 and RW-1 were then periodically gauged from late July 2001 to mid-November 2001 to collect data (recharge rates, volume of free product, groundwater elevation, etc.). In August 2001, URS designed the low-flow liquid pumping system and recovery wells RW-2 & 3 were installed on August 27, 2001 and were included in the periodic gauging events. Materials and equipment for the system were ordered in October and November 2001. Installation and implementation of the system is currently schedule for the weeks of February 11 and 18, 2001.

REMEDIAL ACTION PLAN

The release of MCB has affected the area under an existing tank farm and process area at WGK. Access to the area is also restricted for plant operation health and safety reasons (Level C respiratory protection is required to enter the release area). In addition, the presence of voids in the subsurface underlying concrete driveways and walkways severely restricts the weight loads that can be applied in the area. Because of this restriction, heavy drilling equipment can not be used at this area to install recovery wells. However, from the data collected from the periodic gauging events, it was determined that MCB can be recovered along the western and southern boundaries of the tank farm and process areas as well as from near the source area using RW-1, 2, & 3 and PZ-1, 4, & 9. The boundaries were found to be downgradient of the source area.

Free Product Recovery Plan

Under this plan, free product removal will occur to the extent practicable by pumping liquid MCB from the bottoms of RW-1, 2, & 3 and PZ-1, 4, & 9, when present. The recovery system will utilize the existing nitrogen driven pneumatic diaphragm pump (**Figure 1**). Drop tubes will be placed at the noted recovery wells and piezometers, approximately 3 inches from their respective bottom elevations. This will maximize the recovery of free product and minimize the collection and handling of groundwater. The drop tubes will be attached to a wellhead plate via quick-connects. Teflon tubing will connect each wellhead to the pump. Tee connections will be placed throughout the system to allow the isolation of each wellhead for pumping. During each pumping event, each wellhead will be pumped separately for a duration of time necessary to remove recoverable free product from the recovery well or piezometer. The recovered liquid (free product and groundwater) will be pumped into 55-gallon steel drums for storage. The drums will be situated on a containment pad. Solutia will dispose the recovered free product and groundwater in accordance with the WGK Plant's RCRA program.

After the system has been installed, operation and data collection will occur weekly for the first six weeks. After six weeks, using the data collected, an appropriate operation schedule will be created and implemented. PZ-7 will be gauged after the 20,000-gallon storage tank that is presently situated over it is removed from the area. As well, monitoring well GM-29 will be gauged. Based on the data collected, PZ-7 and GM-29 may be added to the system at a future date.

Free Product Recovery Schedule

The duration of this free product recovery program will be contingent on overall effectiveness of this process. This remedial action focuses on free product removal to the extent practicable. Free product removal will continue as long as removal of free product is effective. Free product removal will be considered effective as long as there is an adequate volume of free product available to be removed at a well or piezometer, approximately a 6-inch column. Groundwater collection will be minimized to the extent possible in order to reduce the generation of contaminated groundwater for treatment.

If you have any questions, please do not hesitate to contact us.

Sincerely,
URS Corporation

Matthew R. Foresman, EIT, GRIT
Environmental Geological Engineer

Anthony R. Mellini, Jr., PG
VP/Operations Manager
Remediation and Operating Services

MRF/mrf

SOLUTIA - 070

WPTD 10114

Control to ECAB
cood response w/
SF D.

Solutia Inc.
575 Maryville Centre Drive
St. Louis, Missouri 63141

P.O. Box 66760
St. Louis, Missouri 63166-6760
Tel 314-674-1000

SOLUTIA
Applied Chemistry, Creative Solutions

February 8, 2002

RECEIVED
FEB 14 2002
DIVISION FRONT OFFICE
Waste, Pesticides & Toxics Division
U.S. EPA - REGION 5

Mr. Robert Springer
Director, Waste, Pesticides and Toxics Division
D-8J
U.S. Environmental Protection Agency, Region V
77 W. Jackson Boulevard
Chicago, IL 60604-3590

Mr. William Muno
Director, Superfund Division
S-6J
U.S. Environmental Protection Agency, Region V
77 W. Jackson Boulevard
Chicago, IL 60604-3590

Re: Solutia's WG Krummrich Plant Corrective Action

Dear Bob and Bill:

I would like to update you on the progress being made with regard to the Corrective Action being undertaken at the Solutia W.G. Krummrich Plant ("WGK Plant") in Sauget, Illinois as well as the CERCLA activity in the Sauget area. As you may recall, Solutia and U.S. EPA entered a RCRA Administrative Order on Consent on May 3, 2000 which had a deadline of January 1, 2002 regarding control of groundwater migration from the WGK Plant to the Mississippi. On December 27, 2001, EPA granted Solutia a three month extension of time to that deadline.

Complicating the attainment of the deadline for control of groundwater migrating from the WGK Plant are the Sauget Area 2 Superfund Sites, as well as other industrial facilities, many of which are located between the Plant and the Mississippi River. We discussed the need for coordination of the RCRA and CERCLA aspects during our meeting in Chicago on October 3, 2001, as reflected in my letters to you of October 31 and December 20 and your letter to me of December 17. Since the October 3 meeting, I believe we have achieved encouraging progress toward implementing the coordinated approach that we all have agreed to pursue.

The Sauget Area 2 Sites and the other industrial facilities are likely impacting the contamination of the groundwater that is migrating to the Mississippi. As we have agreed, the plumes from these different sources are commingled and should be addressed in an integrated manner. In your letter of December 17 you noted our agreement that "an interim response action performed at Sauget Area 2 is the appropriate mechanism."

Last November, the group of PRPs that are undertaking investigatory work of the Sauget Area 2 Sites, received from Mike Ribordy in the CERCLA section of Region V a request for submission of a Focused Feasibility Study (FFS) to address the groundwater that is migrating under Site R toward the River. Solutia, on behalf of the PRP group, submitted a draft FFS on December 21, 2001. The FFS recommends installation of a recovery well system between Site R and the Mississippi in order to create a hydraulic barrier to decrease the effect of the contaminants entering the River which originate from four of the Sauget Area 2 Sites, four of the Sauget Area 1 Superfund Sites and various industrial facilities including the WGK Plant.

Since the submission of the FFS, Solutia has received comments from Mr. Ribordy and Mr. Bardo regarding the design of the recovery wells. I am pleased to report that a review of their comments indicates that we have a high level of common agreement on the essential features of this work. Solutia has not, however, received comments on the other sections of the FFS. It is our understanding that Mr. Ribordy is waiting on comments from the Illinois Environmental Protection ("IEPA"), as well as EPA's contractor on the project, before he can write a draft plan. Once Mr. Ribordy has the information he needs from all the comments, he can make public a draft Interim Plan for addressing this groundwater issue. At least 30 days of public comment on the draft plan must be allowed under the CERCLA statue before the Interim Plan can be finalized.

It is expected, as stated above, that the interim remedy issued under the CERCLA program will be the means of controlling the groundwater from the WGK Plant. Thus, at the point the CERCLA section gets all the comments in, those comments can be forwarded to the RCRA section and a draft plan can be put in place to move forward in a coordinated fashion..

Meanwhile, EPA has granted to Solutia an extension of the deadline for control of groundwater migration to the Mississippi from the WGK Plant until April 1, 2002. Currently, the expectation is that it will be at least mid-March before the public comment period on the CERCLA Interim Plan can be closed. Once that is done, the Agency must write a response to comments and finalize the Plan. The chances that a Final Plan will be in place by April 1 are diminishing while Mr. Ribordy is waiting for the IEPA and EPA contractor comments.

As we have discussed, Solutia will need to obtain further relief from the RCRA division at Region V with respect to the deadline under the RCRA AOC of May 3, 2000 for demonstrating compliance with the Environmental Indicator regarding control of groundwater migration. Since it is EPA's technical judgment that the EI will not be fully satisfied until the groundwater recovery wells referred to above have been installed and put into operation, such extensions will need to cover that full period of time. In particular, we will need to receive a further extension from the currently applicable April 1, 2002 deadline prior to that date.

We are alerting you now to this situation so that we can have a dialogue comfortably in advance of that deadline on how best to address this situation. We look forward to discussing this with you and with members of your staff.

Sincerely,

Brent J. Gilhousen

Brent J. Gilhousen
Assistant General Counsel, Environmental

CC: Ken Bardo
Mike Ribordy

SOLUTIA - 071

G.H.

EW

SOLUTIA

Applied Chemistry, Creative Solutions

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RECEIVED
MAR 05 2002
DIVISION FRONT OFFICE
Waste, Pesticides & Toxics Division
U.S. EPA - REGION 5

March 1, 2002

Mr. Robert Springer
Director, Waste, Pesticides and Toxics Division
D-8J
U.S. Environmental Protection Agency, Region V
77 W. Jackson Boulevard
Chicago, IL 60604-3590

Re: Solutia's WG Krummrich Plant Corrective Action

Dear Bob

This letter is a follow up to my February 8, 2002 letter to you regarding progress being made in Sauget, Illinois. We think it is vital that you continually stay in the loop on this unique and important project. Again I want to express my appreciation for all you have done to facilitate this project so that the Agency and Solutia obtain their goals in getting the Sauget Area issues addressed in a timely, environmentally sound and cost effective manner.

In an effort to continue an open dialogue between Solutia and Region V, as well as continue an open communication between the Region V RCRA and CERCLA programs, a meeting was held at EPA's Chicago office on February 14, 2002 . In attendance were Alan Faust and Bruce Yare of Solutia, Mike Ribordy (Region V, CERCLA program) and Ken Bardo (Region V, RCRA program). These four discussed the progress of the work regarding the Focused Feasibility Study ("FFS") which addresses groundwater discharges to the Mississippi River at the Site R landfill. As you know, the discharges to the River include a commingled groundwater plume from both CERCLA and RCRA sites in the Sauget vicinity.

The CERCLA sites in Sauget are on a slightly different time schedule than the WG Krummrich Plant is, under its RCRA Corrective Action Order. You and Mr. Muno both have acknowledged that this timing difference causes various difficulties. Because of this, and because of the involvement of other PRPs in the CERCLA sites, Region V has agreed to coordination of CERCLA and RCRA interests into one time schedule with CERCLA driving the schedule.

At the meeting in Chicago last week, it was agreed that the following schedule is a likely scenario for the CERCLA program to appropriately process the FFS:

March 21, 2002	Solutia submits revised FFS (revised pursuant to comments from IEPA, EPA and EPA's contractors).
April 1, 2002	EPA (CERCLA) issues a Proposed Plan based on the FFS, the public comment period on the FFS begins.
May 1, 2002	Public comment period closes, EPA begins drafting the response to comments.
June 1, 2002	EPA issues an Interim Record of Decision ("Interim ROD" or "ROD").

After the Interim ROD is finalized, an EPA order can be issued requiring the performance of <u>the</u> work determined to be necessary in the ROD.

We understand that it is EPA's intent that at least Solutia (if not other PRPs) implement the remedy that EPA determines to be necessary under the Interim ROD. You stated in your December 17, 2001 letter to me that compliance with the Interim ROD will satisfy Solutia's obligation pursuant to the RCRA AOC to demonstrate compliance with the Environmental Indicator for control of migration of contaminated groundwater.

Currently, Solutia has until March 31, 2002 to comply with the Environmental Indicator for groundwater. The next step in the CERCLA process is getting an Interim ROD issued. The earliest that will occur is June 1, 2002. It can be expected that an Order to undertake that work will be issued sometime by September 1, 2002. Because it may take until September to finish the necessary CERCLA steps, Solutia asks that the Agency issue it an extension from the current March 31st deadline to September 31, 2002.

Once the order is issued under CERCLA, it is our understanding, based on your December 17th letter, that Solutia will be considered in compliance with the mandate in the RCRA order to demonstrate compliance with the Environmental Indicator for control of contaminated groundwater. If our understanding is incorrect, please let us know.

Sincerely,

Brent J. Gilhousen

Brent J. Gilhousen

cc: Mr. William Muno
Mr. Mike Ribordy
Mr. Ken Bardo

SOLUTIA - 072

MAR 2 0 2002

DE-9J

Via E-mail and First-Class Mail

Mr. Robert Hiller
Solutia Inc.
500 Monsanto Avenue
Sauget, IL 62206-1198

RE: Final Comments on the Groundwater
Discharge Control System
Solutia Inc.
ILD 000 802 702

Dear Mr. Hiller:

The United States Environmental Protection Agency (EPA) has reviewed Solutia's draft *Groundwater Discharge Control System, Design Basis and Design Response to Comments* submitted on January 28, 2002. The response addresses EPA's comments on the *Discharge Control Study* and *Technical Specifications* dated December 27, 2001.

Enclosed are EPA comments that need to be addressed in the final design of the groundwater discharge control system. If you have any questions regarding the enclosed comments, I can be reached at (312) 886-7566 or at bardo.kenneth@epa.gov.

Sincerely yours,

Kenneth S. Bardo

Kenneth S. Bardo, EPA Project Manager
Corrective Action Section

Enclosure

cc: Alan Faust (via E-mail), Solutia
Jim Moore, Illinois EPA
Gina Search, Illinois EPA

bcc: Mike Ribordy, RRB #1
Thomas Martin, ORC
Richard Murawski, ORC
Rick Hersemann, Tetra Tech EMI

DE-9J:KBARDO:6-7566:kb:03/19/02 Solutia Containment Design Final Comments

OFFICIAL FILE COPY

ENCLOSURE

Comments on "Response to Comments [on the] Design Basis"

General Comment

Responses are provided to EPA's technical review comments on the draft "Discharge Control Study"; however, a revised version of the "Discharge Control Study" incorporating the responses was not submitted for review. Ensure that the comment and responses are incorporated into the design basis.

Specific Comments

1. **Response to General Comment No. 1, Sensitivity Analysis, Page 1-5, Note at Top of Page.** The note states that the discharge rate of 650 gallons per minute (gpm) presented for the three extraction wells is an "overestimate" of the discharge rate predicted by Solutia's model. This overestimate is not mentioned in the draft "Discharge Control Study." The model predicted that the discharge rate necessary to prevent contaminated groundwater flow to the Mississippi River is 535 gpm. The discharge rate of 650 gpm includes an increase of 115 gpm to account for "unknowns in the modeling process." The specific unknowns are not identified in Solutia's response. Solutia's note mentions only that the value of 650 gpm was used to "address flow variability issues and modeling unknowns such as the ones indicated by the sensitivity analysis." Provide a detailed explanation for overestimating the discharge rate for the three extraction wells and ensure that the total discharge rate meets the performance objectives of the system and is incorporated into the design basis.

   In addition, the number of extraction wells to be used varies in the studies. The draft "Discharge Control Study" indicates that the barrier will consist of three extraction wells with a total discharge rate of 650 gpm. In contrast, the draft "Mass Containment Study" indicates that model runs were performed using two extraction wells with a total discharge rate of 650 gpm. The hydraulic containment scenario could be significantly affected by the number and configuration of extraction wells used in the hydraulic barrier. Clearly specify the number of extraction wells being proposed for the hydraulic barrier and ensure that the appropriate number and total discharge rate is justified, meets the remedial objectives, and is incorporated into the design basis.

2. <u>Response to General Comment No. 1, Page 1-6, First Full Paragraph</u>. This paragraph states that use of a 650-gpm discharge rate for the groundwater extraction system will capture 85 percent of the mass loading to the Mississippi River. The paragraph also states that the "amount of mass removal needed to achieve remedial objectives and performance standards is not known." However, the response also states that the proposed hydraulic barrier will achieve the remedial objectives included in the Sauget Area 2 focused feasibility study (FFS) submitted to EPA on December 21, 2001. If the amount of mass removal needed to achieve remedial objectives is not known, capturing 85 percent of the mass loading to the Mississippi River may not achieve remedial objectives. Explain how the remedial objectives for the hydraulic barrier will be met by the proposed groundwater extraction system and ensure that the necessary remedial objectives (e.g., demonstrating that the discharge to the river is insignificant or acceptable according to an appropriate interim assessment) are incorporated into the design basis.

3. <u>Response to General Comment No. 1, Page 1-6, Second Full Paragraph</u>. This paragraph discusses installation of a hydraulic barrier to achieve the remedial objectives stated in the FFS. The paragraph states that the hydraulic barrier will provide hydraulic control of affected groundwater. The paragraph also states that the toxicity and volume of groundwater contaminants "will be reduced through the action of natural processes, such as biodegradation, adsorption, dilution, volatilization and chemical reactions with subsurface material, occurring between the source areas and the hydraulic barrier and by removing and treating impacted groundwater migrating to the Mississippi River."

   It is not apparent that such factors as biodegradation, adsorption, dilution, volatilization, and chemical reactions with subsurface materials are significant, or have been studied, at the facility. It is also unclear whether the remedial design was partially based on these potential mechanisms for attenuation of contaminant concentrations. The "Source Evaluation Study" does not discuss adsorption, dilution, volatilization, and chemical reactions as mechanisms for attenuation and only mentions theoretical rates for biodegradation. Provide additional detail on the significance of potential mechanisms for contaminant plume dissolution and the affects they might have on the hydraulic containment of the contaminant plume.

4. <u>Response to General Comment No. 2, Page 1-9, First Paragraph</u>. This paragraph states that the groundwater flow model discussed in the draft "Discharge Control Study" is a steady-state model. The paragraph also states that no transient model was developed because previous transient modeling results reflected only "minor changes" from steady-state modeling results. These "minor changes" are not identified in the response. Provide additional detail on the differences between the transient and steady-state modeling results and ensure that the groundwater model incorporates appropriate hydraulic conditions into the design basis.

5. <u>Response to Specific Comment No. 1, Page 1-10</u>. The response states that the "Shallow Hydrogeologic Unit" acts as a confining layer. It is unclear whether an actual confining layer is present between the shallow and deep horizons or the low horizontal and vertical hydraulic conductivities of the shallow horizon allow it to act as an aquitard. Specify the technique used to model the upper saturated zone and the confining layer between the upper and middle saturated zones, if such a layer is present.

<u>Comments on "Response to Comments [on the] Design"</u>

General Comment

A number of Solutia's responses to EPA's technical review comments state that the design specifications and drawings have been corrected based on the comments; however, the revised specifications and drawings have not been submitted for EPA review. Ensure that the revised specifications and drawings are incorporated into the design basis.

Specific Comments

1. <u>Response to Specific Comment No. 1, Page 2-1</u>. The response states that specification Section 2.2.1 will be revised to read as follows: "A. 10-inch I.D. Type 304 stainless steel pipe with flush threaded joints and Teflon 'O' rings." The proposed revision should also include the thickness of the pipe.

   The original specifications called for use of "low carbon stainless steel" for well casing material. Type 304 stainless steel called for in the proposed revision is not considered to be "low carbon stainless steel." Clarify this discrepancy.

2. <u>Response to Specific Comment No. 2, Page 2-1</u>. The response states that the language in Part B of specification Section 2.2.2 is intended to provide some quality control (QC) capacity into the grout mixing process. However, it is not clear how QC can be implemented in this case because the specification Section 2.2.2, Part A, clearly states that neat cement grout will consist of "cement and water in proportion of 1 bag (94 lb) Type 1 Portland cement to 8.3 gal clean water." The specification does not include any provision to alter this mix ratio. Clarify the nature of the QC capacity intended.

3. <u>Response to Specific Comment No. 5, Page 2-3</u>. The response does not specify the thickness of the discharge tubing required for well pump discharge pipe installation. It is not clear whether the discharge tubing's thickness has been added to the relevant drawings because they were not submitted for review. Ensure that the revised drawings addressing this comment are incorporated into the design basis.

4. <u>Response to Specific Comment No. 14, Page 2-9</u>. The response states the following: "Placement of check valves in [the] vault will cause backflow through the pumps in the event of pump shutdown. This backflow may shorten the life of the pump particularly if the pump is energized while the water is flowing down the drop pipe and through the pump impellers." Typically, when such a pump is energized, water cannot flow backward unless the power supply phases are reversed. Such a phase reversal should not be a problem because the electrical contractor is supposed to check all motors for proper rotation. Also, a well pump is usually designed to handle backflow. The rate of backflow is typically controlled with air and vacuum release valves or with pump control valves installed between the pump and the check valve. This type of installation allows access to the check valve for maintenance and reduces the water hammer force. Review the design for potential backflow problems based on these considerations. Also, surge relief valves should be used to protect the piping.

5. <u>Response to Specific Comment No. 17, Page 2-10</u>. The response does not take into consideration the problems associated with use of level switches in extraction well applications. Level switches are typically set at predetermined elevations within a well. The switches cannot be easily adjusted to compensate for seasonal groundwater level fluctuations or fluctuations caused by significant storms, flooding, or other wells being out of service. A better alternative is use of a radio frequency (RF) level control or a capacitance probe level control.

With this type of control, the probe can span the entire depth of the water column or any portion of it, and the set points can easily be adjusted at the control panel. An RF level control also provides groundwater readings for the well that can be used to readjust various set points such as the high level alarm and the pump stop level. Consider revising the extraction well control scheme based on these considerations.